HYDRAULIC BEHAVIOUR OF SIDE-WEIRS

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MASTER OF TECHNOLOGY

by

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To

my Mother and Father

CERTIFICATE

This is to certify that the thesis entitled 'HYDRAULIC BEHAVIOUR OF SIDE-WEIRS' by SUBHASH CHANDRA AWASTHY is record of work carried out under my supervision and has not been submitted elsewhere for a degree.

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NOMENCLATURE

A	area of the flow section
B	width of the channel
B_W	width of the water surface at Weir level
C	a coefficient
C_{II}	a coefficient
C_a	a coefficient
C_b	a coefficient
C_d	a coefficient
C_M	a coefficient
C_{M^*}	a constant coefficient
D	diameter of the pipe
D	hydraulic depth
E	specific energy
E_W	specific energy with reference to Weir Crest level
F	Froude number
F_1	upstream Froude number
F_2	down stream Froude number
f_1	a function
fn	a function
fn'	a function
g	acceleration due to gravity
H	total head
h_1	upstream depth over the weir crest
h_2	downstream depth over the weir crest

K	a constant
L	length of the side-weir
n	rugosity coefficient
p	an exponent
Q	flow through a section
Q_1	upstream flow
Q_2	downstream flow
Q_a	available flow
Q_b	flow in the channel when the weir starts functioning
Q_s	total flow over the side-weir
Q_o	upstream design discharge
q	an exponent
q_s	discharge per unit width over the side-weir
R	hydraulic radius
r	an exponent
S_f	friction slope
S_o	bed slope
s	height of the weir crest
T	top width of the water surface
V	mean velocity at any point
V_1	upstream mean velocity
V_c	critical velocity corresponding to upstream flow
V_J	velocity of the jet
x	distance along the side-weir measured from upstream end of the weir

y	depth at any point
y_1	upstream depth measured at the centre line of the channel
y_2	downstream depth measured at the centre line of the channel
y_c	critical depth corresponding to upstream flow
y_n	normal depth in the upstream channel
Z	section factor
z	height of the bed of the channel above a datum
α	energy coefficient
β	momentum coefficient
θ	angle of the jet
θ'	inclination of the jump front with the normal to the centre line of the channel
$\emptyset$	varied flow function

ABSTRACT

An extensive experimental study has been done on the behaviour of side-weirs in the rectangular channels. The spatially varied flow equation of De Marchi for the flow along the side-weir has been found to be useful for the determination of discharge over the side-weirs. The parameters affecting the coefficient of discharge have been identified. In subcritical flow, an expression has been derived analytically for the variation of the discharge coefficient of a side-weir of zero height with the Froude number of upstream main flow. The experimental data show a very good agreement with this expression. In supercritical flow the discharge coefficient is virtually independent of any of the flow parameters. The discharge coefficient of side-weirs of finite height has been studied experimentally and is shown to be essentially the same as for the corresponding side-weir of zero height.

An empirical equation similar to ordinary weir formula has been developed for easy solution of discharge over the side-weirs and has been found to give the accuracy of $\pm$ 10% upto upstream Froude number of 0.5.

The hydraulic jump occuring along the side-weir has been studied in an exploratory way.

CHAPTER I

INTRODUCTION

A side-weir is defined as a free overflow weir set into the side of a channel with the purpose of allowing part of the liquid to spill over the side if the surface of the flow in the channel rose above the side-weir crest. It is also called as side-spillway or lateral spillway. As contrary to the function of ordinary weir which blocks up the flow in the main channel, the function of a side-weir is to facilitate the overflow.

Some of the situations in hydraulic engineering where side-weirs are in use are:

a) In sanitary engineering practice the side-weir is used extensively as stormflow outlet in the combined sewer system, to pass a chosen proportion of the storm water to some convenient river, stream or estuary, at the earliest possible moment so as to reduce the cost of the sewer system.

b) In an irrigation canal system the surface runoff may sometimes be let into a canal and excess flow may be disposed off at some convenient location downstream to some other canal, river, stream or estuary. The various structures which could be used for this purpose are (i) Bottom-rack (ii) sluice gate set into the

side (iii) side-weir. Both the bottom-rack and the sluice gate have been mainly used as sediment extractors. In case the interest lies in passing out top layers of the main flow having less sediment content, provision of a side-weir will be advantageous.

c) The lateral spillways have been used to protect flood plain embankments from overtopping at the time of floods.

d) In the hilly regions the intake of the diversion canals may be situated in a deep and narrow gorge. Circumstances may require provision of a side-weir in the head works. Such was the situation in the case of head works of the Ouse-Great Lake canal taking off from Ouse-river in U.S.A. (8).

e) A branch canal or distributory at its take-off point usually has a sluice gate fixed over a sill for the control of the discharge. When the sluice gate is fully lifted, water spills over freely and in such a case the structure should be recognised as a free overflow side-weir. The discharge into the tributory in such a case will be as a result of side-weir action.

f) Water from gutters of residential streets is sometimes diverted to subsurface drains by means of kerb-opening inlets. When the slot inlet is partially submerged, the structure should be recognised as a free over flow side-weir set into the side of a channel of triangular cross-section.

g) In the recent years side-weir has also found some use in thermal power installations. After cooling the power plant, sometimes the warm water is carried in a channel to be spread over a long length of the pond with the use of a side-weir.

Fig. 1.1 is a definition sketch in which various parameters appearing in the side-weir problem have been defined. For the known initial flow (V_1, y_1) and the geometry of side-weir (L, B, s), the problem of side-weir consists of:

i) prediction of discharge division (Q_s/Q_1), and

ii) prediction of water surface in the vicinity of the side-weir.

Till the year 1928 there existed a few empirical formulae for calculating the flow over a side-weir. A theoretical approach to the problem was given by Nimmo (8) in the year 1928 and DeMarchi(3) in the year 1932. For spatially varied flow with withdrawal of water from sides and with certain assumptions regarding specific energy (E) and weir action DeMarchi derived the following equation for the variation of depth along the side-weir in rectangular channel (3).

$$x = \frac{3}{2} \frac{B}{C_M} \left[\phi \left(\frac{y}{E}\right) + k \right] \qquad \ldots (1.1)$$

where

$$\phi\left(\frac{y}{E}\right) = \frac{2E-3s}{E-s}\sqrt{\frac{E-y}{y-s}} - 3 \sin^{-1}\sqrt{\frac{E-y}{E-s}} \qquad \ldots (1.2)$$

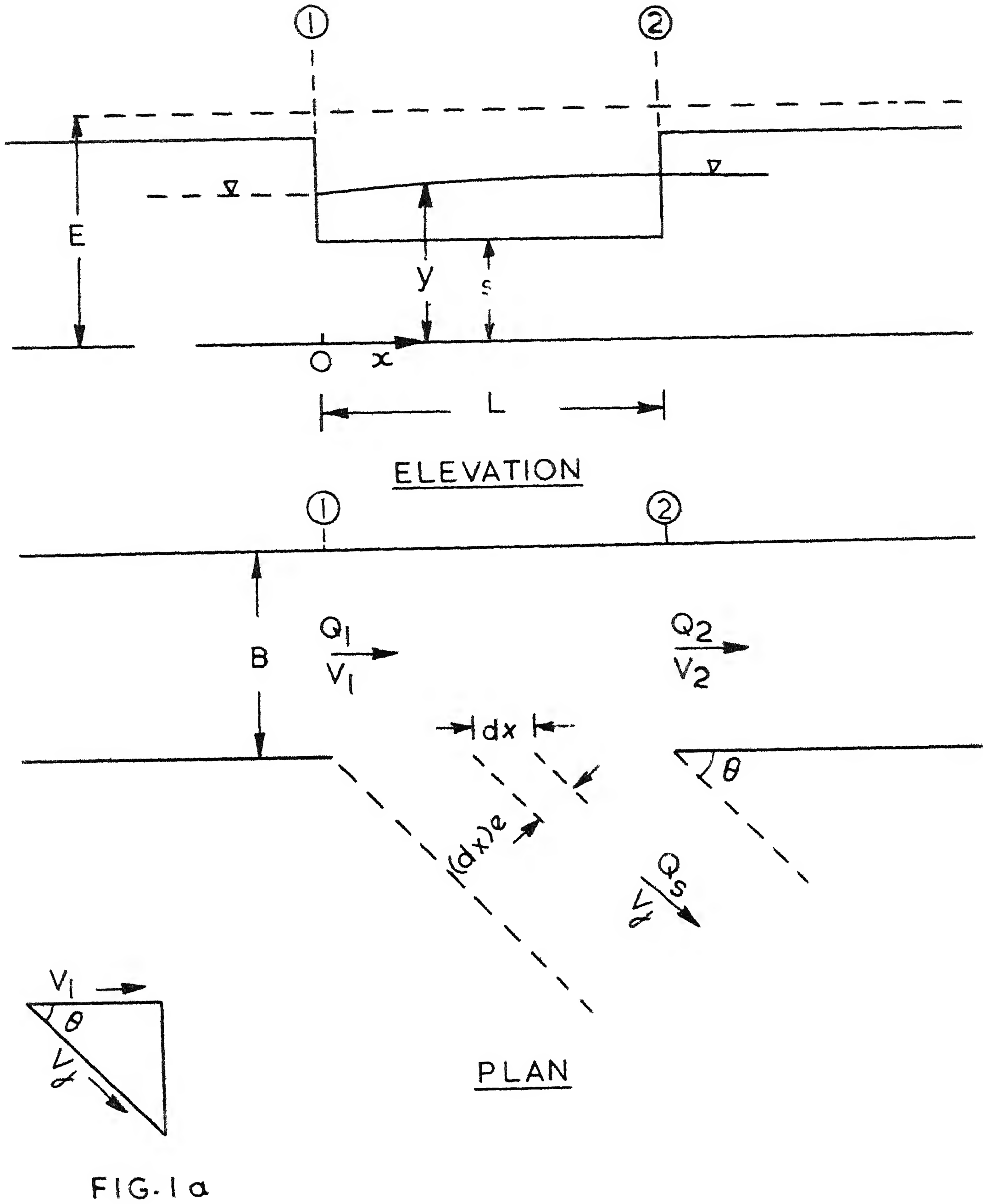


FIG. 1.1 DEFINITION SKETCH

$E = y + \frac{V^2}{2g}$ = specific energy, constant along the weir

x = distance along the weir

y = depth of water at any point

V = mean velocity at any point in x direction

s = height of weir crest above channel bottom

C_M, k are constants

B = width of the channel

g = acceleration due to gravity (Ref. Fig. 1.1)

Further in the year 1957 there were a few exploratory works which were either not much different from the previous works or empirical in nature.

The present state of knowledge on side-weirs is insufficient to explain fully the mechanics of flow. Even though it is found that DeMarchi theory could be satisfactorily applied for predicting the flow profile and the discharge division, the knowledge about the behaviour of the experimental coefficient (C_M) is lacking.

The present investigation was undertaken with the following objectives:

i) to have a better understanding of the flow mechanism.

ii) to study the variation of the coefficient of discharge (C_M) with other parameters of the problem.

iii) to predict the discharge division (Q_s/Q_1), if possible, by a simpler procedure.

The details of the investigation have been presented in Chapters III, IV, and V. A detailed study of the existing literature which formed a basis for the present study is presented in the next chapter.

CHAPTER II

LITERATURE REVIEW

It appears that the pioneering work on the problem of side-weirs has been done in the field of public health engineering in the early years of the 20th century. Since then this problem has attracted the attention of many investigators who have evolved various theories and formulae giving the relationships between quantity, depths of flow and lengths of side-weirs. A brief review of the some of the empirical formulae has been presented in table I.

A rational approach to the problem of side-weirs was given by DeMarchi (3) in the year 1932, based on the following assumptions:

a) The flow is unidirectional

b) The velocity distribution across the channel section is constant and uniform; that is, the velocity distribution coefficients are taken as unity. However, proper values of the coefficients may be introduced, if necessary.

c) The pressure variation in the flow is hydrostatic; that is, pressure distribution coefficients are taken as unity.

d) The slope of the channel is relatively small, so its effects on the pressure head and on the force on the channel sections are negligible.

Table I: Existing Empirical Formulae

Sl. No.	Authors	Reference	Formulae	Units	Remarks
1.	Engels	6	$Q_s=3.32L^{.83}(y_2-s)^{1.67}$	F.P.S.	Large scale model, not useful for design
			$Q_s=3.32L^{.9}(y_2-s)^{1.60}$	F.P.S.	Width gradually reduced
2.	Fruhling	4	$Q_s = 4\ L\ h_1^{1.5}$	F.P.S.	Gives high discharge
3.	Parmley	4	$L=\frac{B_w V}{1.67}\left(\sqrt{\frac{1}{h_2}}-\sqrt{\frac{1}{h_1}}\right)$	F.P.S.	For $h_2=0$, L is ∞ Always falling profile
4.	Coleman and Smith	4	$L=29.06\ B^{1.4}h_1^{0.513}$		
			$Q_s=1.674\ BL^{.72}h_1^{1.645}$	F.P.S.	Always falling profile
5.	Babbit	6	$L=2.3V_1.D.\mathrm{Log}_{10}\left(\frac{y_1-s}{y_2-s}\right)$	F.P.S.	Always falling profile

Where,

D = diameter of the pipe

V_1 = upstream velocity

h_1 = upstream depth over the weir

B_w = width at the weir level

V = velocity at any point along the weir.

e) The flow may be treated as flow diversion where the diverted water does not affect the energy head, i.e. the specific energy along the side-weir is constant; that is,

$$E = y + \frac{V^2}{2g} = \text{specific energy, constant along the side-weir} \quad \ldots \quad (2.1)$$

where, y, V are depth and mean velocity at any cross-section.

f) The ordinary weir formula

$$q_s = \frac{2}{3} \; C_M \sqrt{2g} \; (y-s)^{3/2} \quad \ldots \quad (2.2)$$

is applicable, where

C_M = a discharge coefficient

s = the height of the weir crest.

The total energy (H) at a channel section is given by:

$$H = z + y + \alpha Q^2/2gA^2 \quad \ldots \quad (2.3)$$

*where, Q = discharge through any cross section along the weir

z = distance of the bottom of the channel section above a horizontal datum.

A = Area of the flow section

Differentiating equation (2.3) with respect to x,

$$\frac{dH}{dx} = \frac{dz}{dx} + \frac{dy}{dx} + \frac{\alpha}{2g} \left(\frac{2Q}{A^2} \frac{dQ}{dx} - \frac{2Q^2}{A^3} \frac{dA}{dx} \right) \quad \ldots \quad (2.4)$$

and noting that $\frac{dH}{dx} = - S_f$, $\frac{dz}{dx} = - S_o$, $\frac{dQ}{dx} = q^*$,

$$\frac{dA}{dx} = \left(\frac{dA}{dy} \right)\left(\frac{dy}{dx} \right) = T \frac{dy}{dx}$$

the equation (2.4) is reduced to

$$\frac{dy}{dx} = \frac{S_o - S_f - \alpha Q q^*/gA^2}{1 - \alpha\ Q^2/gA^2 D} \qquad \ldots \qquad (2.5)$$

where $D = A/T$

Equation (2.5) is the dynamic equation for spatially varied flow with decreasing discharge.

Since the specific energy (E) along the side-weir is assumed constant, i.e. $S_f = S_o$ and for horizontal channel $S_o = 0$, taking $\alpha = 1.0$, for rectangular channel equation (2.5) is reduced to:

$$\frac{dy}{dx} = \frac{Qy\ (-dQ/dx)}{gB^2y^3 - Q^2} \qquad \ldots \qquad (2.6)$$

$$-\frac{dQ}{dx} = q_s = \frac{2}{3}\ C_M \sqrt{2g}\ (y-s)^{3/2} \qquad \ldots \qquad (2.7)$$

From equation (2.1)

$$Q = By\sqrt{2g(E-y)} \qquad \ldots \qquad (2.8)$$

From equations (2.6, 2.7 and 2.8)

$$\frac{dy}{dx} = \frac{4}{3} \cdot \frac{C_M}{B}\ \frac{\sqrt{(E-y)(y-s)^3}}{3y-2E} \qquad \ldots \qquad (2.9)$$

On integrating,

$$x = \frac{3}{2} \cdot \frac{B}{C_M}\ \phi\left(\frac{y}{E}\right) + \text{constant} \qquad \ldots \qquad (2.10)$$

where

$$\phi\left(\frac{y}{E}\right) = \frac{2E-3s}{E-s}\sqrt{\frac{E-y}{y-s}} - 3\sin^{-1}\sqrt{\frac{E-y}{E-s}} \qquad \ldots \qquad (2.11)$$

ϕ (y/E) is a varied flow function.

For a side-weir of length L,

$$L = \frac{3}{2} \frac{B}{C_M} (\phi_2 - \phi_1) \qquad \ldots \qquad (2.12)$$

where suffix 1 and 2 refer to beginning and end of the weir.

Collinge (5) plotted DeMarchi function $\phi(y/E)$ against y/E for different values of s/E for easy solution of equation (2.12). He found experimentally that the discharge coefficient (C_M) decreases with increase in mean velocity along the weir.

Ackers (1) derived an equation which is similar to DeMarchi equation. He considered the momentum and energy coefficients which were considered to be unity by DeMarchi, and obtained

$$L = \frac{B}{C} \sqrt{2g/\alpha} \left[\left\{ \left(2 - \frac{\beta s}{E_w}\right) \left(\sqrt{\frac{E_w}{h_2} - \beta} - \sqrt{\frac{E_w}{h_1} - \beta} \right) \right\} - \left\{ 3\sqrt{\beta} \left(\mathrm{Cos}^{-1} \sqrt{\frac{\beta h_2}{E_w}} - \mathrm{Cos}^{-1} \sqrt{\frac{\beta h_1}{E_w}} \right) \right\} \right] \qquad \ldots \qquad (2.13)$$

where,

$E_w = h + \frac{V^2}{2g} =$ constant

h = depth above the weir crest

α = Energy coefficient

β = momentum coefficient

C = a constant $\quad (C = \frac{2}{3} C_M \sqrt{2g})$

h_1 = upstream depth above the weir crest

h_2 = downstream depth above the weir crest

As a result of experimental investigations he found that $\alpha = 1.4$, $\beta = .8$, $C = 3.33$ (f.p.s. units) and $h_1 = \frac{1}{2} E_w$

For the special case in which h_2 is taken as 1.9 cms (1/16 ft.), he derived a simplified equation for the design of the side-weir.

$$L = 2.03\ B\ (5.28 - \frac{2.63\ \bar{s}}{E_w}) \quad \text{in f.p.s. units} \qquad (2.14)$$

This formula was stated to be applicable for the case of **falling** profile which he found would occur if $s/E_w < 1$.

It should be noted here that the reliable information on the value of C_M corresponding to various flow conditions is lacking. Ackers (1) suggests a value of $C_M = 0.625$ if y is measured at a remote distance from the plane of the weir and $C_M = 0.725$ if y is measured at the plane of the weir. Apparently C_M is assumed to be constant by him. Collinge (5) finds that C_M varies with the mean velocity of the main channel.

Allen (2) in connection with his work on side-weirs in circular pipes found that:

$$\frac{Q_s}{Q_a} = C_a\ (L/D)^{2/3} \qquad \ldots \qquad (2.15)$$

where

$$C_a = 0.22\ D/B_w \quad \text{(empirical)}$$

B_w = width of the water surface at weir level

Q_a = available flow

(The flow corresponding to the depth equal to sill height is not available for flow over the weir. Unavailable flow subtracted from total flow gives the available flow.)

Frazer's(6) approach to the problem is simple semi-empirical curve fitting. He derived correlation equations for following cases:

(i) Rapid Flow: Correlation of quantity and depth of flow

$$q_r = y_r \sqrt{2.5 - 1.5\, y_r} \quad \ldots \quad (2.16)$$

where

$$q_r = Q/Q_1$$

$$y_r = y/y_c$$

y_c = critical depth corresponding to initial flow

(ii) Rapid flow: Correlation of quantity and length

$$q_\infty = C_r \sqrt{2.5 - 1.5\, C_r} \quad \ldots \quad (2.17)$$

where

q_∞ = discharge in the downstream channel

$C_r = s/y_c$

$$\frac{1-q_r}{1-q_\infty} = 1 - 10^{-L/8B} \quad \ldots \quad (2.18)$$

The left hand side is the ratio of the discharge over the weir in the length $l = L/y_c$ to the discharge over a weir of the same proportionate height whose length is infinite.

(iii) Subcritical Flow: Correlation of quantity and depth of flow:

$$q_r = y_r \sqrt{0.99\ \theta_1 - 2y_r} \qquad \ldots \qquad (2.19)$$

where,

$$\theta_1 = 2y_{1,r} + \frac{1}{(y_{1,r})^2}$$

$$y_{1,r} = y_1/y_c$$

(iv) Subcritical Flow: Correlation of quantity and length

$$Q_1 - Q_2 = \frac{2}{3} C_b \sqrt{2g}\ (\bar{y} - s)^{1.5}\ L \qquad \ldots \qquad (2.20)$$

or $\quad 1-q_2 = C_{II}\ (\bar{y}_r - s_r)^{1.5} \cdot x_r$

where,

$$q_2 = Q_2/Q_1$$

$$x_r = L/B$$

$$\bar{y}_r = (2y_{2,r} + y_{1,r})/3$$

$$y_{2,r} = y_2/y_c$$

C_b = constant, $\quad C_{II} = \dfrac{2/3 \cdot C_b \cdot \sqrt{2g}}{Q_1}$

Experimentally he found that

$$C_{II} = 0.73 - \frac{0.32}{y_{1,r}} - \frac{0.14}{l} \qquad \ldots \qquad (2.21)$$

$$l = L/y_c$$

Krishnappa and Seetharamiah (7) made use of DeMarchi equation for predicting the flow in a 90° branch channel with subcritical flow in the main and supercritical flow in the branch channel. However in applying the equation they have used the heads measured directly above the weir. They found that a defined coefficient of discharge is a function of F_1 and L/B, where F_1 is the upstream Froude number. The experimental range both for F_1 and L/B was 0 to 1. It may be noted that the analogy of side weir of zero height as a branch channel flow, used by Krishnappa and Seetharamiah (7) is not correct in view of the differences in aeration, friction and downstream confinement of flow in the two cases.

After a study of the existing literature as above the following comments could be made:

a) The previous work is not sufficient to explain the mechanics of flow.

b) The empirical formulae have dimensional constants which may be true for the experiments backing them. However, by their very nature, they cannot be of general use.

c) Though complexity is introduced due to draw-down, splashing and non-uniformity of the flow as observed by Frazer (6), still the equation for spatially varied flow (equation 2.12) appears to be a rational and better approach to the problem.

d) The reliable information on the coefficient of discharge (C_M) is lacking.

e) A cross section of the channel will show a H_2 type water surface profile. In applying DeMarchi equation (2.12) the selection of the right depth is a problem. Use of depth over the weir, as done by Krishnappa and Seetharamiah (7) is not a happy choice, as the depth over the weir suffers from the defects of (a) curvature of the stream lines (b) the pressure is different ~~than~~ from hydrostatic. Probably the centre line depth will be a better depth to use.

Chapter III deals with the experimental details of the investigation which was carried out with a view to improve the existing state of knowledge on side-weirs.

CHAPTER III

EXPERIMENTS

3.1 Introduction

An experimental study of the problem of side-weir in a prismatic horizontal rectangular channel was carried out at the hydraulics laboratory of the Indian Institute of Technology, Kanpur.

The justifications for the investigation were:

a) The existing literature is insufficient,

(i) to explain clearly the mechanics of flow along the side-weir ,

(ii) to predict the nature of water surface in the immediate vicinity of the side-weir.

b) An extensive study of the variation of DeMarchi coefficient of discharge (C_M) with other parameters is lacking.

c) DeMarchi equation is rather tedious to use in connection with side-weirs of finite height. So the investigation was also aimed at finding a simple and sufficiently reliable method of predicting discharge over side-weirs.

3.2 Experimental Set-up

The main experiments were conducted in a 9.0 metre long and 61 cms. wide masonary channel with horizontal bed, specially constructed for the present investigation. Using a precision level the bed of the channel was levelled accurately upto third decimal

place of a centimetre. The schematic view of the masonry channel is shown in Fig. 3.1. For furture reference this flume has been called as flume A. Verifications of some of the observations on flume A were done on a wooden flume hereafter called as flume B. Flume B was about 3 m long, 24.8 cms. wide and the bed was about 1.5 metres above the ground level. The width of the flume B was reduced to 12.4 cms. by fixing a wooden plank at the centre line of the channel in certain experiments.

3.2.1 <u>The Side-Weir</u>

On one side of the masonry flume (flume A) a slot of 1 metre length was cut and an iron frame with three legs was embedded in the wall, with bottom leg being about 3 cms. below the bed of the channel. An aluminium plate was so fixed to the frame that it was flush with the inside surface of the wall. The weirs of required length and height could be conveniently cut and sharp bevelled edges made by filing. The details of a side-weir are shown in Fig. 3.1. The upstream end of the weir was fixed at 4.5 m from the upstream sluice gate. In flume B on which the studies were confined to the side-weir of zero height and subcritical flow, the side was cut and bevelled to form a weir.

3.2.2 <u>The Supply</u>

The water was fed into the channel by three pumps drawing water from an under ground tank and delivering a maximum discharge of about 70 litres per second. Later the supply was doubled by connecting a supply pipe from the over head tank.

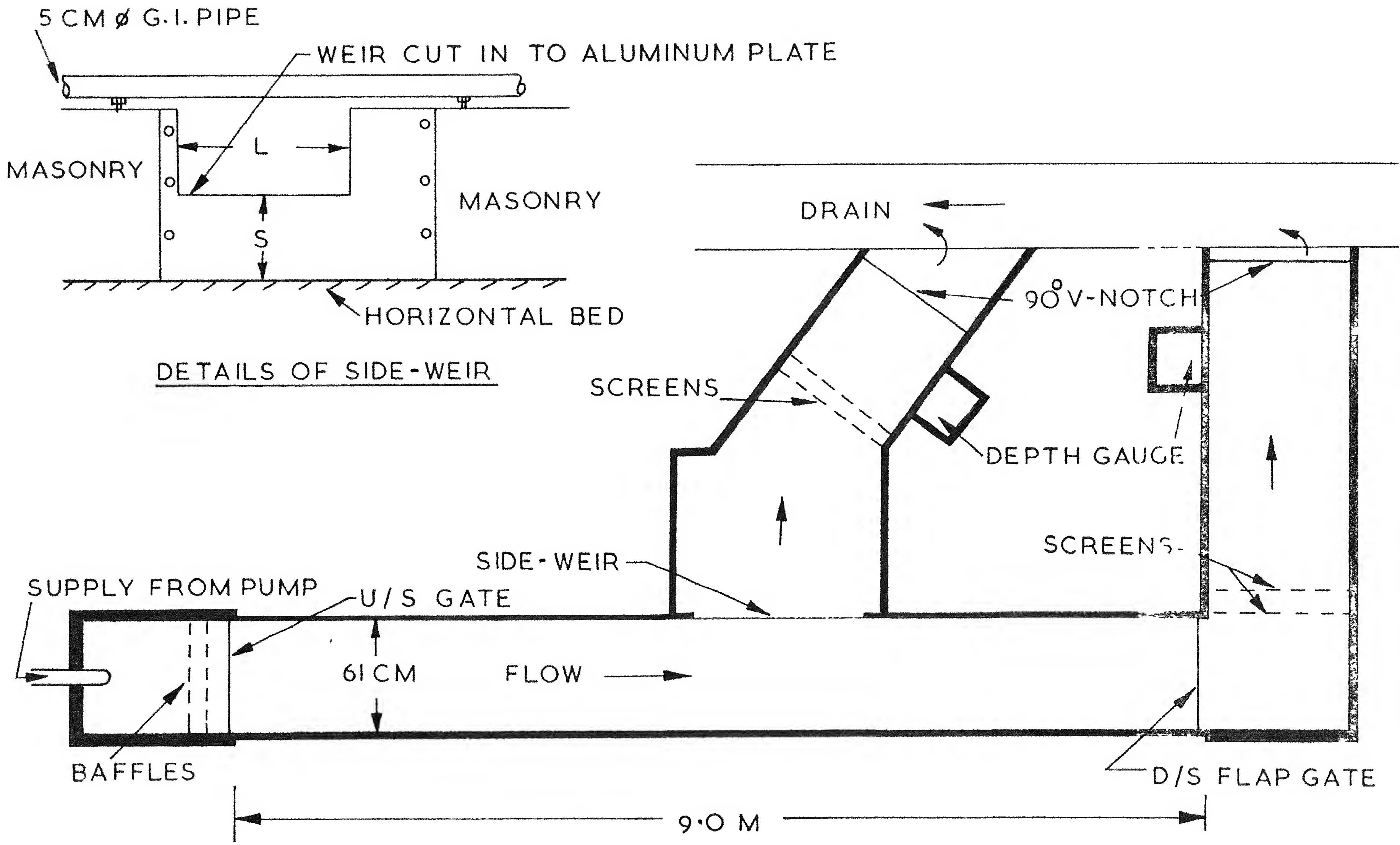


FIG. 3.1 SCHEMATIC VIEW OF EXPERIMENTAL SET-UP

3.2.3 Measurements

The bed of the flume A was about 0.5 m above the ground level. Both the outgoing flow (Q_2) and the flow over the weir(Q_s) were received into the channels 1 m. wide. The screens were provided to make the flows uniform. The uniform flows were allowed to pass over the 90° V notches before delivering to the drain. Stilling chambers were provided for the measurement of depths over the V-notches. In the flume B where maximum supply from the pump was about 8 litres/second the flow rates were measured volumatrically. Depths in the channel were measured by point gauges accurate upto .001 ft. (.03 cms.) and fixed on an iron angle capable of being moved freely over 5 cm. dia. G.I. pipes fixed horizontal over the sides of the channel. In the flume B the velocities were measured by Prandtl type pitot tube and the readings were recorded on an inclined manometer.

After a thorough study of the existing literature it was found that a complete discription of the flow phenomenon occuring along the weir is lacking. Many runs were made for the water surface and velocity profiles along and across the side-weir. The inspection of the flow was also carried out using the dye and some sand. The study of the movement of the dye and sand particles gave knowledge about the possible places of scour and silting near the side-weir in a sandy channel.

3.3 Water Surface Profiles

A series of runs was made with gradually increasing flows in the main channel. For each run the depth of water on the centre line was measured. The measurement of depths on the centre line was decided upon after preliminary readings had shown the effect of draw-down by the weir to be negligible at the centre line of the channel. The water surface profile at a cross-section taken across the weir was of . H_2 type. The typical water surface profiles have been shown in Fig. 3.2. It could be seen that the depth profile in the plane of the weir is highly uneven. Any formula based on depth measurements over the weir should be expected to give considerable errors.

3.4 The Velocity Profiles

The typical velocity profiles have been shown in Fig. 3.3. It will be seen from the plot of mean velocity that the side-weir affects the flow conditions considerably upstream. The approaching uniform flow gets accelerated when it reaches near the upstream end of the weir. The energy coefficient α, is of the order of 1.03 and momentum coefficient β is approximately 1.01. But while applying the DeMarchi equation the error due to omission of α and β may be negligible.

3.5 Flow Separation

A problem with the normal outlet like side-weir lies in the possibility of flow separation which results in unfavourable

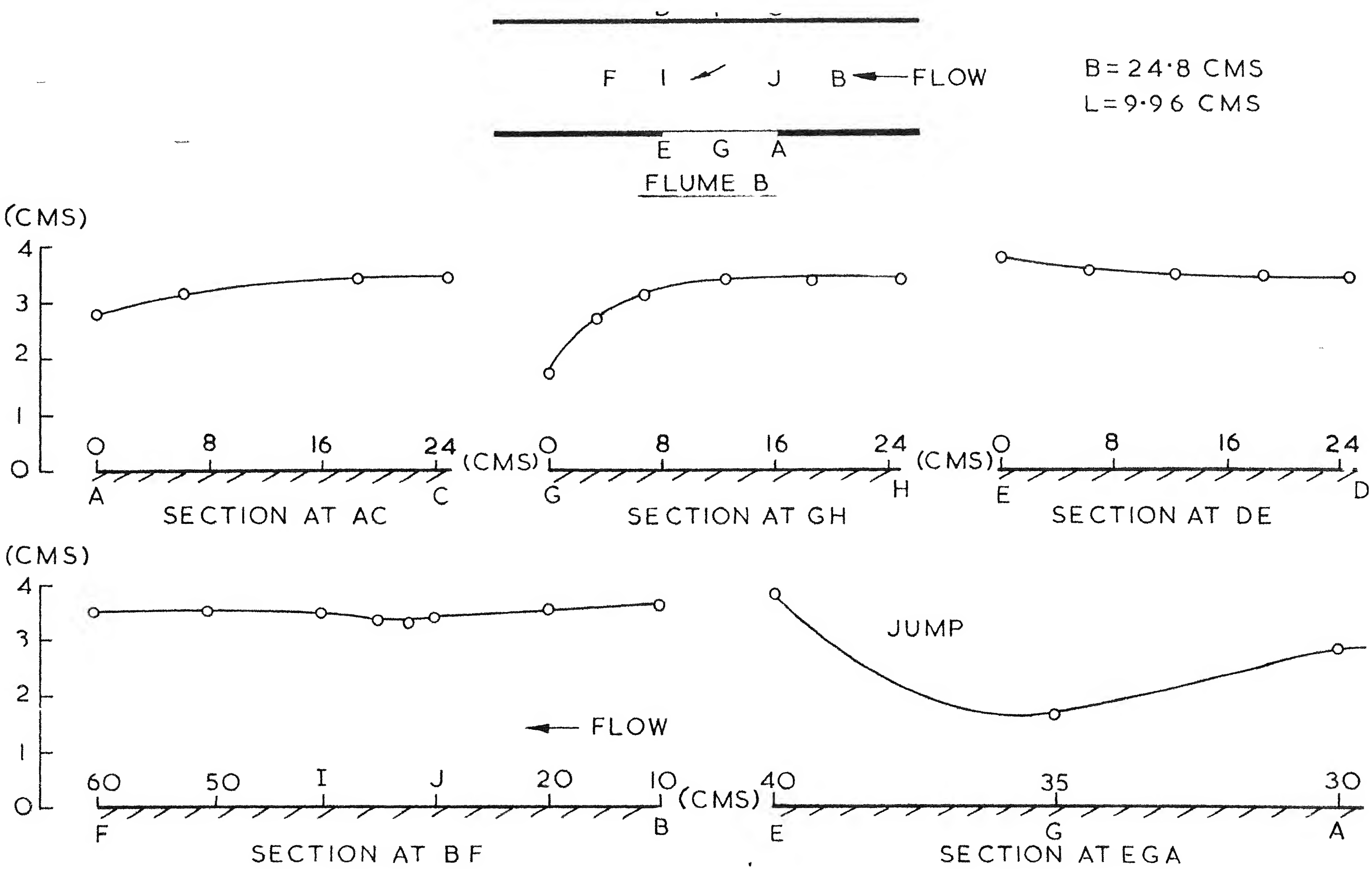


FIG. 3 2 TYPICAL WATER SURFACE PROFILES
(Expt. no.B-38) (Subcritical flow)

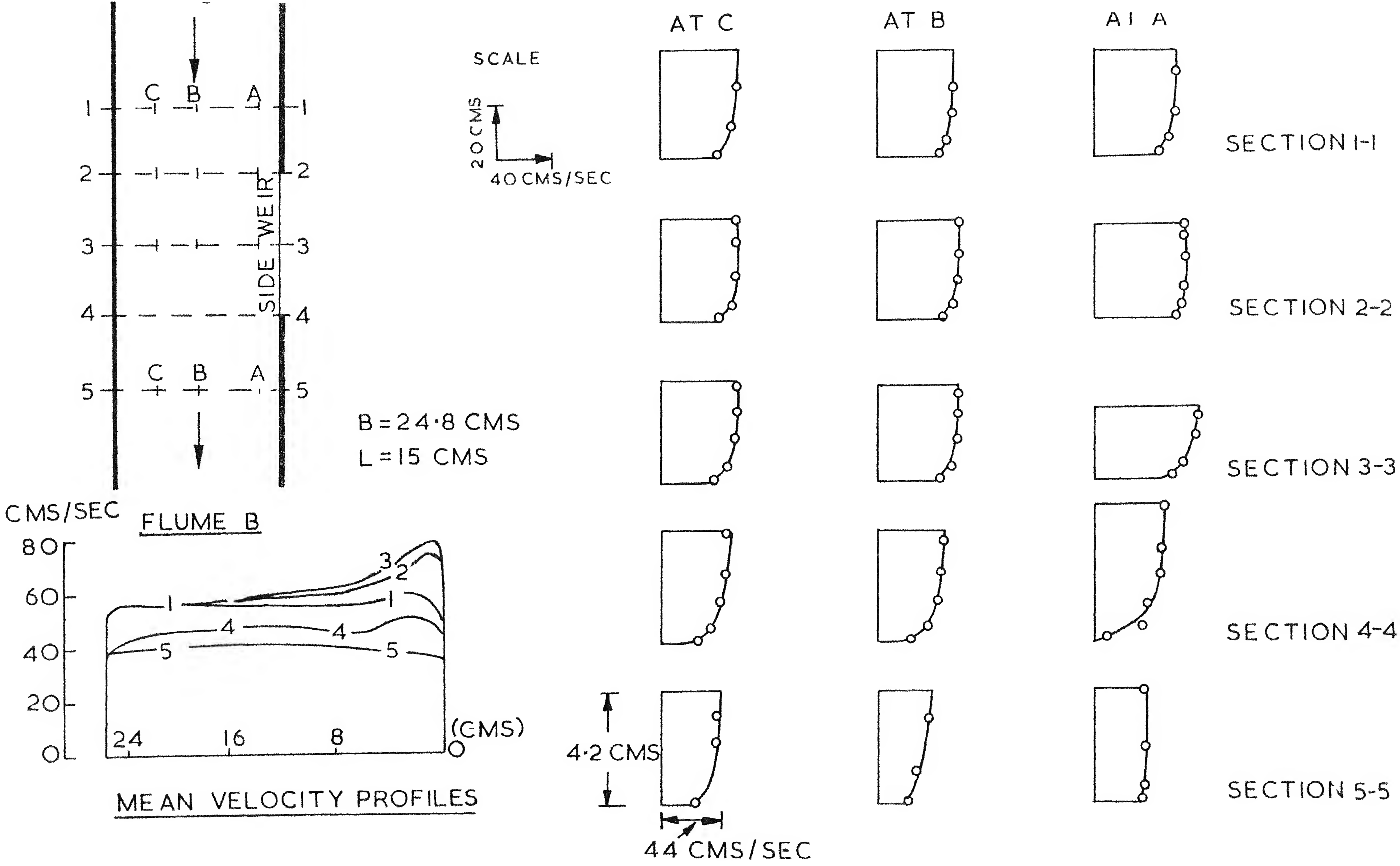


FIG. 3.3 TYPICAL VELOCITY PROFILES (Expt. no. B-45 B)
(Subcritical flow)

pressure gradients and distortion of velocity profiles. The flow coming out of the channel produces a suction effect, as a result the stream lines are curved towards the side-weir. There is tendency of flow separation on the side of channel opposite to the side-weir. The location of the separation depends on

a) the quantity of flow passing over the weir,

b) the velocity in the channel

c) the length of the weir. Once the separation occurs a pocket of back flow is formed which extends further into downstream channel. The size of the back flow pocket is bigger with low Froude numbers. The observations have shown that in case of weirs of finite height the point just downstream end of the weir is susceptible to flow separation, as the flow at that point seems to move up trying to separate from the bed. In case of channels with sandy bed, provision of a smooth transition in the width may possibly solve the problem of silting resulting due to formation of back flow pocket.

3.6 Discharge Division

The efficiency of the side-weir may be defined as the ratio of the flow over the weir to the upstream discharge, i.e. Q_s/Q_1. A total of 200 experiments belonging to 10 series were conducted to study the efficiency of the side-weirs. The corresponding data have been presented in appendix A. The range of

various parameters studied are shown in Table II.

Table II: Range of Parameters Studied

Parameter	Range
F_1	0.02 - 4.3
L/B	0.2 - 1.0
s/y_1	0.2 - 0.96
y_1/L	0.1 - 2.4

For the weirs of zero height the efficiency of the weir for different L/B ratios has been plotted against upstream Froude number (F_1) in Fig. 3.4. The efficiency of the side-weir (Q_s/Q_1) decreases with increase in Froude number (F_1).

The further analysis of the data has been presented in Chapters IV and V.

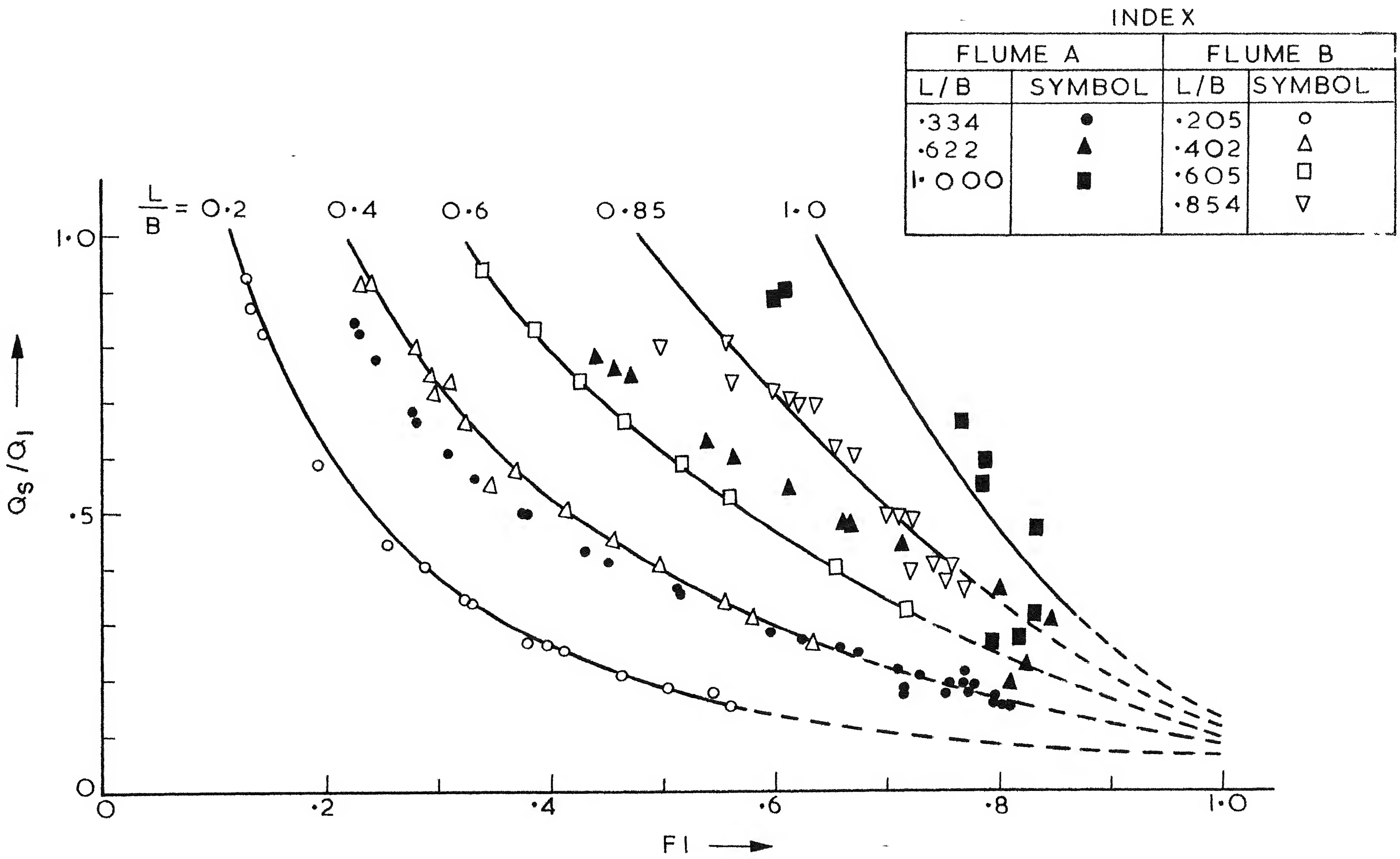


FIG. 3.4. PLOT OF F_1 Vs Q_s/Q_1 FOR DIFFERENT L/B RATIOS OF SIDE-WEIR OF ZERO HEIGHT

CHAPTER IV

ANALYSIS

4.1 Existing Formulae

In the first stage of the analysis of the experimental dat it was found necessary to ascertain the validity and applicabilit of the existing formulae. In Fig. 4.1 the percentage error given by some of the well known existing formulae is plotted against th upstream Froude number (F_1). It will be seen that the percentage error increases with increase in the upstream Froude number F_1. It should be noted that the value of the discharge coefficient (C_M) in DeMarchi equation was assumed constant and equal to .611, due to lack of information. In the other formulae the depth should probably be measured over the weir. But in the calculation of percentage error centre line depths were used. However, it is quite clear that the discharge division is a function of the upstre Froude number. Any formula giving discharge over the side-weir should have upstream Froude number as dominant parameter.

4.2 De Marchi Equation

It was concluded in Chapter II that the best rational approach to the problem seems to be that which is given by De Marchi, Writing here again the De Marchi equation (2.12)

$$L = \frac{3}{2} \frac{B}{C_M} (\phi_2 - \phi_1) \qquad \ldots \qquad (4.1)$$

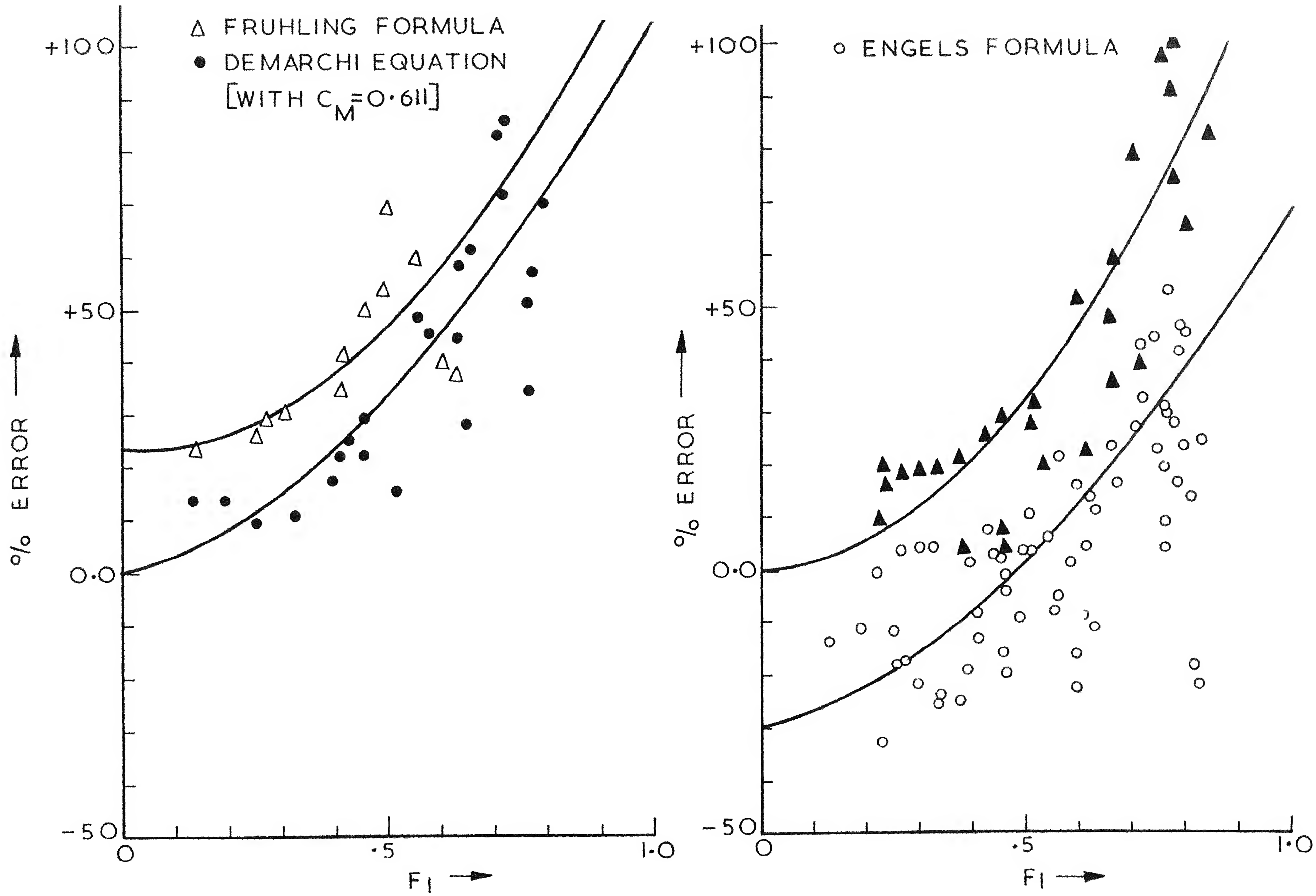


FIG. 4.1 PLOT OF FROUDE NUMBER (F_1) Vs % ERROR BY VARIOUS FORMULAE

where suffix 1 and 2 refer to beginning and end of the weir. If conditions at section 1 (ref. Fig. 1.1) are known (i.e. Q_1, y_1) ϕ_2 and hence Q_2 and y_2 can be found, provided C_M is known. The total discharge over the side-weir, Q_s would then be

$$Q_s = Q_1 - Q_2 \qquad \ldots \qquad (4.2)$$

4.3 De Marchi Coefficient, C_M

It was observed in Chapter II that reliable information on the value of C_M corresponding to various flow conditions is lacking. It is needed to analyse its behaviour based on some theoretical reasoning.

Dimensional analysis indicates that

$$C_M = \text{fn.}\left(F_1 = \frac{V_1}{\sqrt{gy_1}}, \frac{L}{B}, \frac{y_1}{L}, \frac{s}{y_1} \right) \qquad (4.3)$$

It should be expected that F_1 would be a significant parameter affecting the value of C_M and the remaining parameters, representing the geometrical configuration would be having small effects, if any.

For a side-weir of zero height (i.e. $s = 0$).

$$C_M = \text{fn}\left(F_1, \frac{L}{B}, \frac{y_1}{L}\right) \qquad \ldots \qquad (4.4)$$

Assuming L/B and y_1/L have very insignificant effect, an expression for the variation of C_M with F_1, for a weir of zero height is derived as follows.

4.4 Variation of C_M with F_1 for Side-Weir of Zero Height

The flow over the side-weir can be considered as a deflected jet, as in Fig. 1.1. Consider an elemental length dx of the weir. According to equation (2.2)

$$Q_s' = qs \cdot dx = \frac{2}{3} C_M \sqrt{2g} \; dx \cdot y^{3/2}$$

$$(\because \; s = 0) \qquad \ldots \qquad (4.5)$$

Since the effective width of the jet through this element is reduced such that $(dx)_e = dx. \sin\theta$ the discharge Q_s' can be rewritten as

$$Q_s' = \frac{2}{3} \; C_M^* \; \sqrt{2g} \; dx \; \sin\theta \; y^{3/2} \qquad (4.6)$$

where C_M^* is a constant coefficient. Also, as a first approximation, in subcritical flow

$$\sin\theta = \sqrt{1 - (V_1/V_j)^2} \qquad \ldots \qquad (4.7)$$

Now it is assumed that the critical depth corresponding to qs occurs at the plane of the side-weir of zero height such that

$$\frac{V_c^{\,2}}{2g} = \frac{1}{3} E \qquad \ldots \qquad (4.8)$$

where $$E = y_1 + \frac{V_1^{\,2}}{2g} \qquad \ldots \qquad (4.9)$$

substituting $V_c = V_J$ and noting $F_1^{\,2} = \dfrac{V_1^{\,2}}{gy_1}$

$$\sin\theta = \sqrt{1 - \frac{3F_1^{\,2}}{F_1^{\,2}+2}} \qquad (4.10)$$

where C_M^* can be assumed to be 0.611 as it represents the efflux from a constriction with $F_1 \rightarrow 0$, with this

$$C_M = 0.611 \sqrt{1 - \frac{3F_1^2}{F_1^2+2}} \qquad \ldots \quad (4.11)$$

Eqn. 4.11 could be expected to give the variation of C_M with F_1 for subcritical flow over a side-weir of zero height.

In supercritical flow, however, the discharge coefficient C_M could be assumed to be essentially independent of the Froude number as the zone of influence of the side-weir will be confined to its immediate neighbourhood only.

4.5 <u>Experimental Study</u>:

a) Subcritical Flow: Fig. 4.2 shows a plot of C_M, calculated from experimental data for side-weirs of zero height, plotted against the upstream Froude number of the flow. Only data for subcritical region is shown in this plot. Also plotted are the variation of C_M given by Eq. 4.11. It can be seen that all the plotted points follow the variation given by Eq. 4.11 very well. It can be seen that there is no effect of the parameter L/B. A further examination revealed a similar insignificant effect of the parameter y_1/L. Thus it is concluded that the expression given by Eq. 4.11 correctly predicts the variation of C_M with Froude number for subcritical flow in side-weirs of zero height.

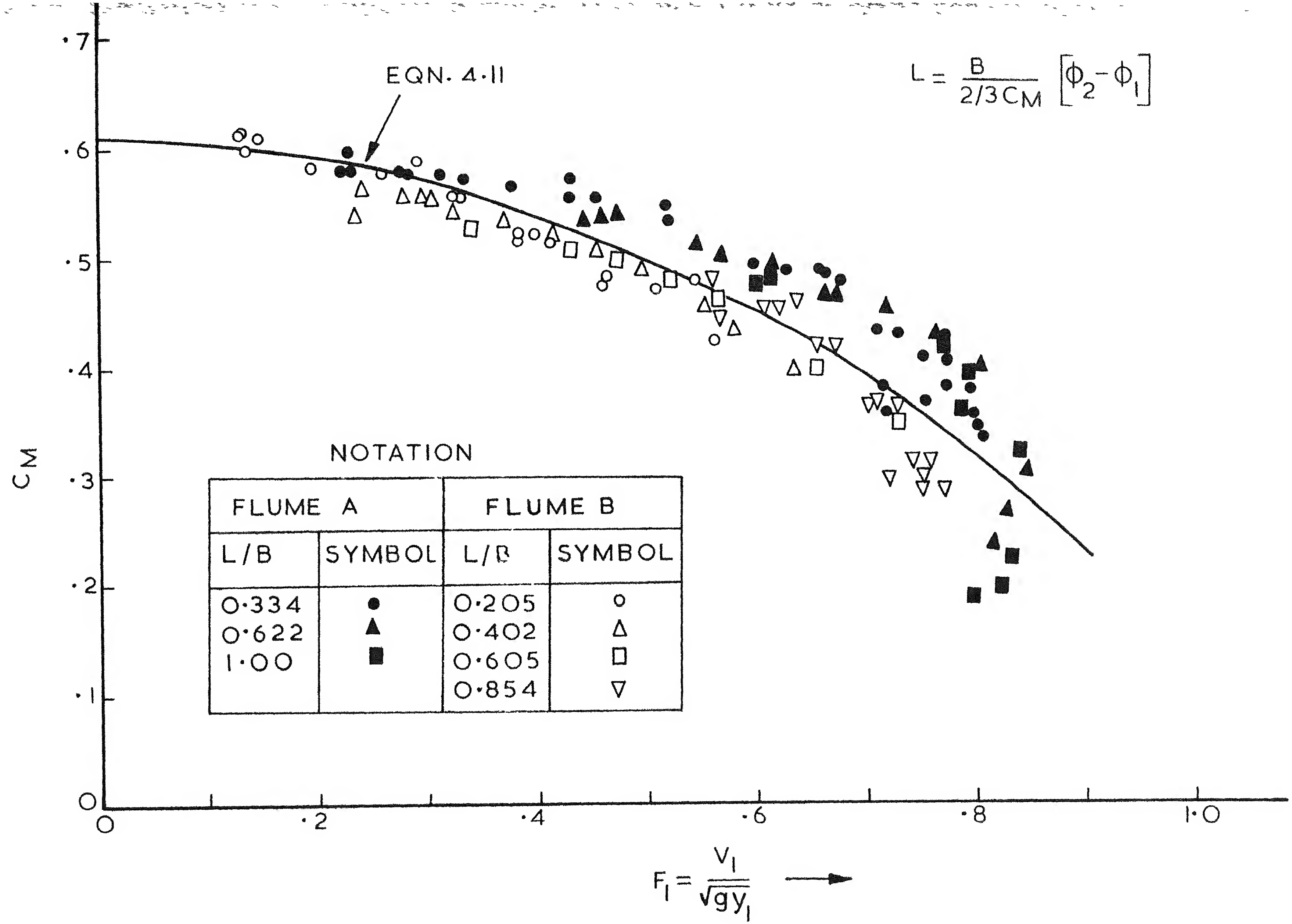


FIG. 4.2. VARIATION OF C_M WITH F_1 FOR SIDE-WEIR OF ZERO HEIGHT

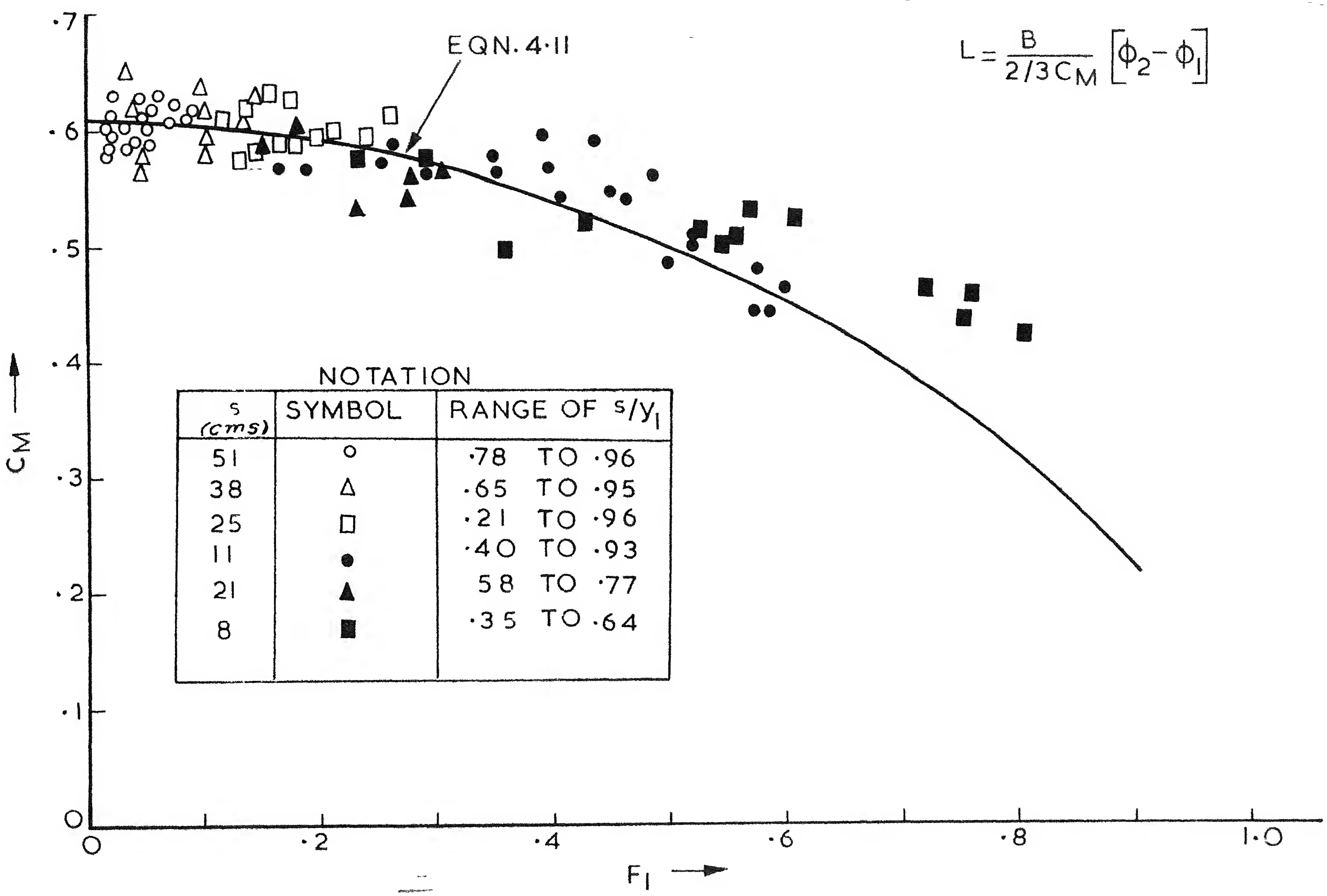
Fig. 4.3 represents the plot of the experimental data in subcritical flow over side-weirs of finite height. A very good fit of the experimental data with the line given by Eq. 4.11 is indicated. It is interesting to note the absence of any significant effect of the parameter s/y_1 and also of other two parameters L/B and y_1/L. A slight observable deviation of data at a Froude number of 0.7 can be attributed to experimental errors.

b) Supercritical Flow: Fig. 4.4 shows a plot of C_M vs. F_1 for supercritical flow. It is once again seen that there is an absence of the effect of the parameters L/B, s/y_1 and y_1/L. Also the effect of Froude number F_1 itself is very small, the variation being expressible as

$$C_M = 0.36 - 0.08 F_1 \quad \ldots \quad (4.12)$$

It is possible that in an ideal case C_M would be independent of F_1. The small effect of F_1, seen in Fig. 4.4 is probably due to friction effects.

It could be concluded that the coefficient of discharge (C_M) in subcritical flow is necessarily a function of Froude number, given by equation 4.11 The effect of any other parameters, if any, is negligible. However, C_M is nearly independent of Froude number or any other parameter in supercritical flow, and its value being equal to 0.36 on the average.



s (cms)	SYMBOL	RANGE OF s/y_1
51	○	·78 TO ·96
38	△	·65 TO ·95
25	□	·21 TO ·96
11	●	·40 TO ·93
21	▲	58 TO ·77
8	■	·35 TO ·64

FIG. 4.3. VARIATION OF C_M WITH F_1 FOR SIDE-WEIRS OF FINITE HEIGHT

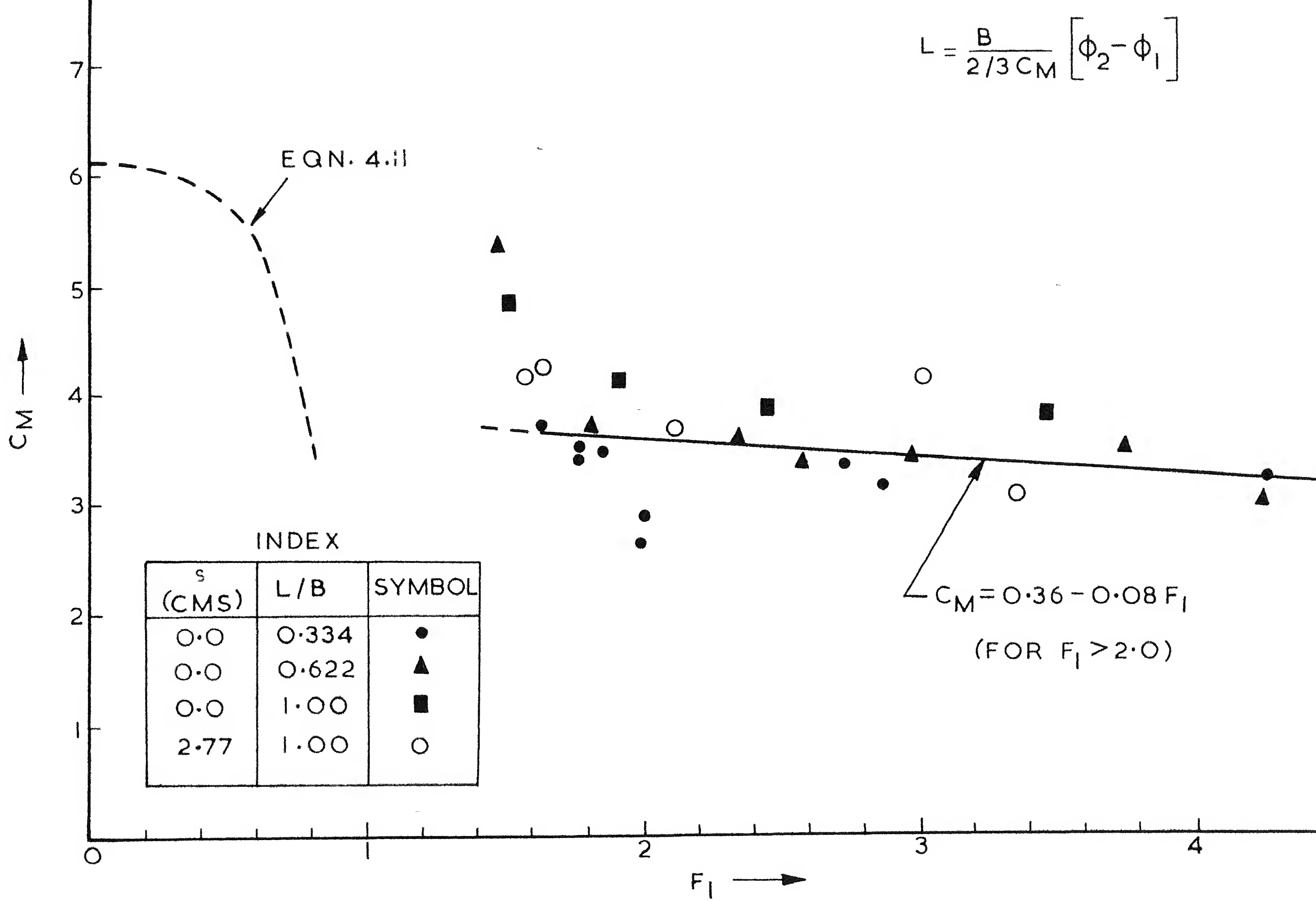


FIG. 4.4 VARIATION OF C_M WITH F_1 IN SUPERCRITICAL FLOW

CHAPTER V

EMPIRICAL DISCHARGE EQUATION

5.1 Design Problems:

Appendix B illustrates the procedure for solving two typical design problems met in the field application of side-weir. A design chart (B-1) of varied flow function ϕ plotted against Froude number (F) for different values of s/y ratio has been developed for easy solution of De Marchi equation, viz

$$L = \frac{3}{2} \cdot \frac{B}{C_M} (\phi_2 - \phi_1) \tag{5.1}$$

The coefficient of discharge C_M has been calculated on the basis of information presented in Chapter IV, i.e.

$$C_M = 0.611 \sqrt{1 - \frac{3F_1^2}{F_1^2+2}} \quad \text{for } F_1 < 1.0$$

and

$$C_M = 0.36 - 0.08 F_1 \quad \text{for } F_1 > 2.0$$

where F_1 is already known for any design problem.

In the first type of design problem Q_1, y_1, s are given and length L of the side-weir is to be designed for a design overflow Q_s. The procedure involves the solution of following cubic equation for y_2 by trial and error.

$$E = y_2 + \frac{Q_2^2}{2g.B^2.y_2^2} \tag{5.2}$$

where,

$$Q_2 = Q_1 - Q_s \qquad (5.3)$$

Once y_2 is known, ϕ_2 and hence L can be calculated.

In the second type of design problem Q_1, y_1, s, L are given and Q_s is to be found. From the given quantities and using De Marchi equation (5.1) ϕ_2 can be calculated. The procedure to find y_2 from known ϕ_2 again involves a trial and error calculation which is tedious. If y_2 is known Q_2 (from Eqn. 5.2) and hence Q_s (from Eqn. 5.3) could be calculated.

The labour involved in trial and error procedure could be reduced if Q_s could be calculated approximately by some other method, say by empirical formula.

5.2 Empirical Formula:

It would be very convenient to calculate Q_s, if it could be expressed by a simple discharge formula involving L, y_1, and s. With this in view, Q_s is expressed by the following Eq.(5.4) which is very similar to ordinary weir formula:

$$Q_s = C_d \frac{2}{3} \sqrt{2g} \; . \; L \; . \; (y_1 - s)^{3/2} \qquad (5.4)$$

in which C_d is a coefficient which takes into account the,

a) curvature effect of the stream-lines

b) velocity and pressure distribution

c) approaching velocity head

d) side contractions

e) variation of depth along the weir.

Dimensional analysis indicates that

$$C_d = Fn \left(F_1 = \frac{V_1}{\sqrt{gy_1}}, \frac{L}{B}, \frac{h_1}{L}, \frac{s}{y_1}\right) \quad \ldots (5.5)$$

where $h_1 = (y_1 - s)$

5.2.1 Subcritical Flow:

By arguments similar to the one proposed for the variation of C_M in section 4.4, it could be shown that for small values of y_1/L, L/B and $s/y = 0$,

$$C_d = f_1(F_1) = 0.611 \sqrt{1 - \frac{3F_1^2}{F_1^2+2}}$$

in subcritical flow (5.6)

However, for finite values of s/y_1 and large values of L/B and y_1/L, the effect of these parameters on C_d could be expected to be significant.

Figs. 5.1 and 5.2 show the variation of C_d with F_1 in subcritical flow for weirs of zero height and for weirs of finite height. The equation (5.6) is also shown plotted in these figures. The close examination of the plotted points revealed that the scatter in these figures is mainly due to the effect of the parameter h_1/L. The effect of other parameters L/B and s/y_1 is not very apparent.

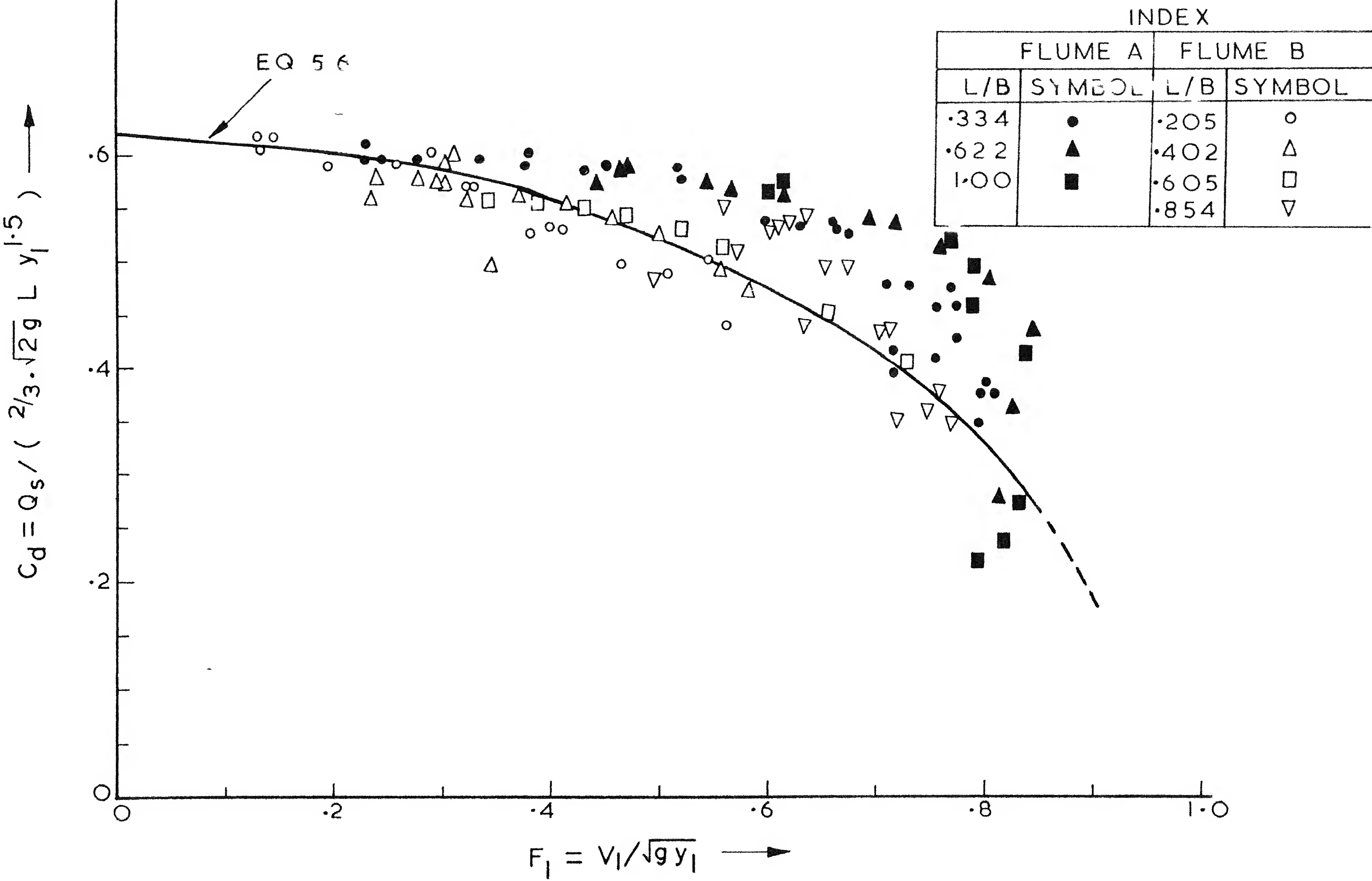


FIG.5·1 VARIATION OF C_d WITH F_1 FOR SIDE-WEIR OF ZERO HEIG

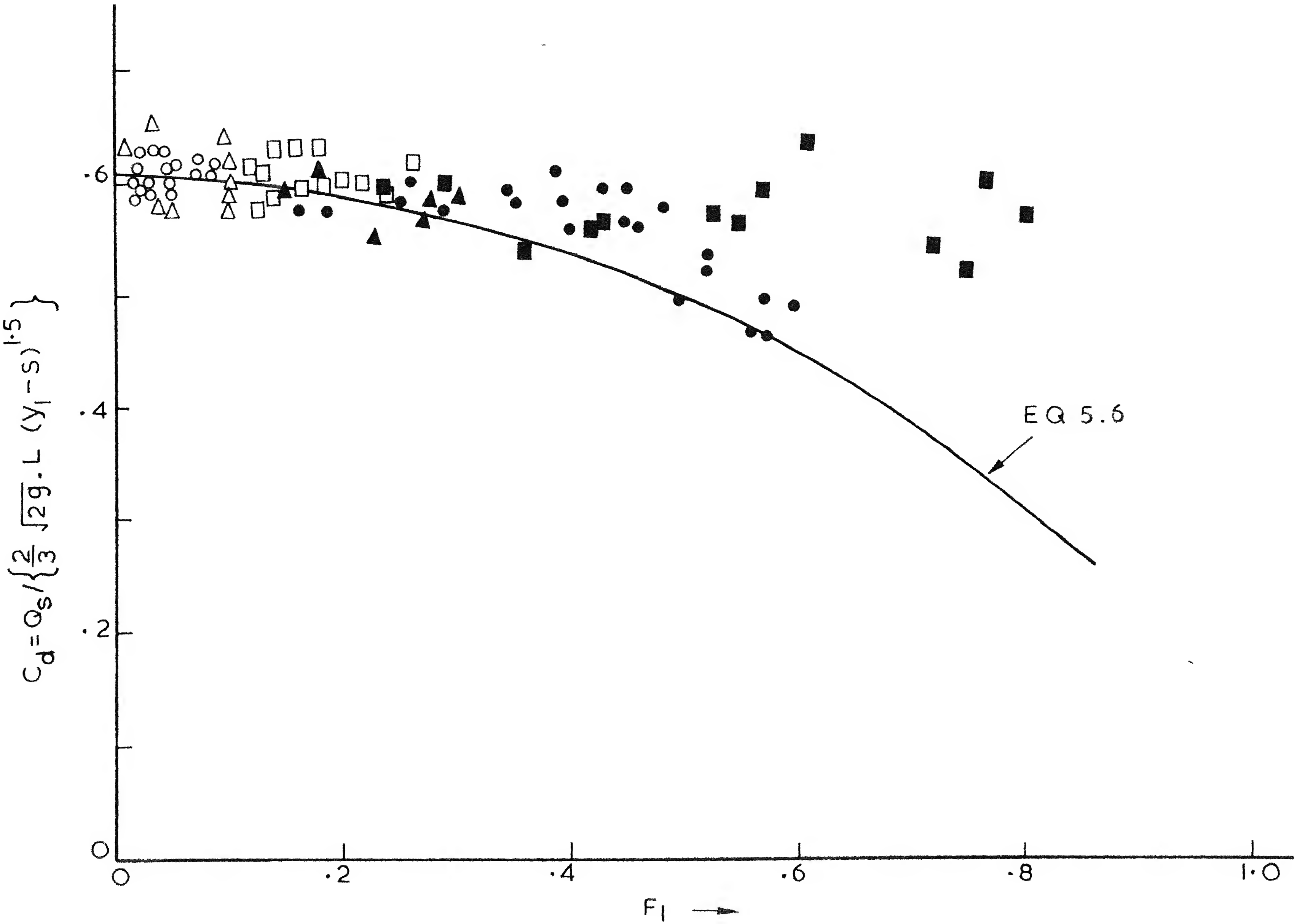


FIG. 5·2. VARIATION OF Cd WITH F_1 FOR FINITE HEIGHT OF WEIRS

The variation of C_d for subcritical flow is expressed as

$$C_d = f_1(F_1) \left(\frac{h_1}{L}\right)^p \left(\frac{L}{B}\right)^q \left(\frac{s}{y_1}\right)^r \qquad \dots (5.7)$$

Or

$$f_1(F_1) = 0.611 \sqrt{1 - \frac{3F_1^2}{F_1^2+2}} = \frac{C_d}{(h_1/L)^p (L/B)^q (s/y_1)^r} \qquad \dots (5.8)$$

From the experimental data for weirs of zero height as well as for weirs of finite height values of p,q,r which gave least variance from the curve of $C_d = f_1(F_1)$ (Eqn. 5.6) were found with the aid of digital computer. It was found that the effect of the parameters L/B and s/y_1 were insignificant (i.e. $q \doteq 0$, $r \doteq 0$). The value of p was found to be 0.06. Hence the discharge equation in subcritical flow could be taken as

$$Q_s = C_d \cdot \frac{2}{3} \sqrt{2g} \cdot L\, (y_1 - s)^{3/2}$$

where

$$C_d = \left(0.611 \sqrt{1 - \frac{3F_1^2}{F_1^2+2}}\right) \times \left(\frac{y_1 - s}{L}\right)^{.06} \qquad .(5.9)$$

The formula (5.9) though empirical has the advantage of simplicity. From this approximate value of Q_s can be calculated very easily. It is necessary to verify this formula (5.9) with the experimental results to ascertain the amount and nature of the error. It could be expected that the error in the calculation of Q_s (% Error) will be more with high Froude numbers. The

percentage error has been plotted against upstream Froude number (F_1) in Fig. 5.3. It could be seen that the error is within $\pm$ 10% band upto $F_1 = 0.55$ (only 9 out of 98 points fall slightly out of this band). For higher Froude numbers the error is large. This could possibly be due to the experimental errors as at high Froude numbers the depth was small and water surface was wavy.

It is concluded that equation (5.9) may be used for the approximate estimate of side-weir discharge for small Froude numbers, (i.e. $F_1 < 0.5$) with the accuracy of $\pm$ 10%.

5.2.2 <u>Supercritical Flow</u>:

It was observed in section 4.5(b) that the coefficient of discharge C_M is virtually independent of upstream Froude number (F_1). Like C_M, coefficient C_d should also be expected to be independent of F_1, i.e. fairly constant in supercritical regime.

Fig. 5.4 shows the variation of C_d with F_1 in supercritical flow. The data of weir of zero height closely follow a straight line variation,

$$C_d = .34 - 0.01\ F_1 \qquad (5.10)$$

showing a small effect of Froude number. Also plotted in this figure are four points for side-weir of finite height (s/y_1 = .23 to .48). Though there is some scatter these data

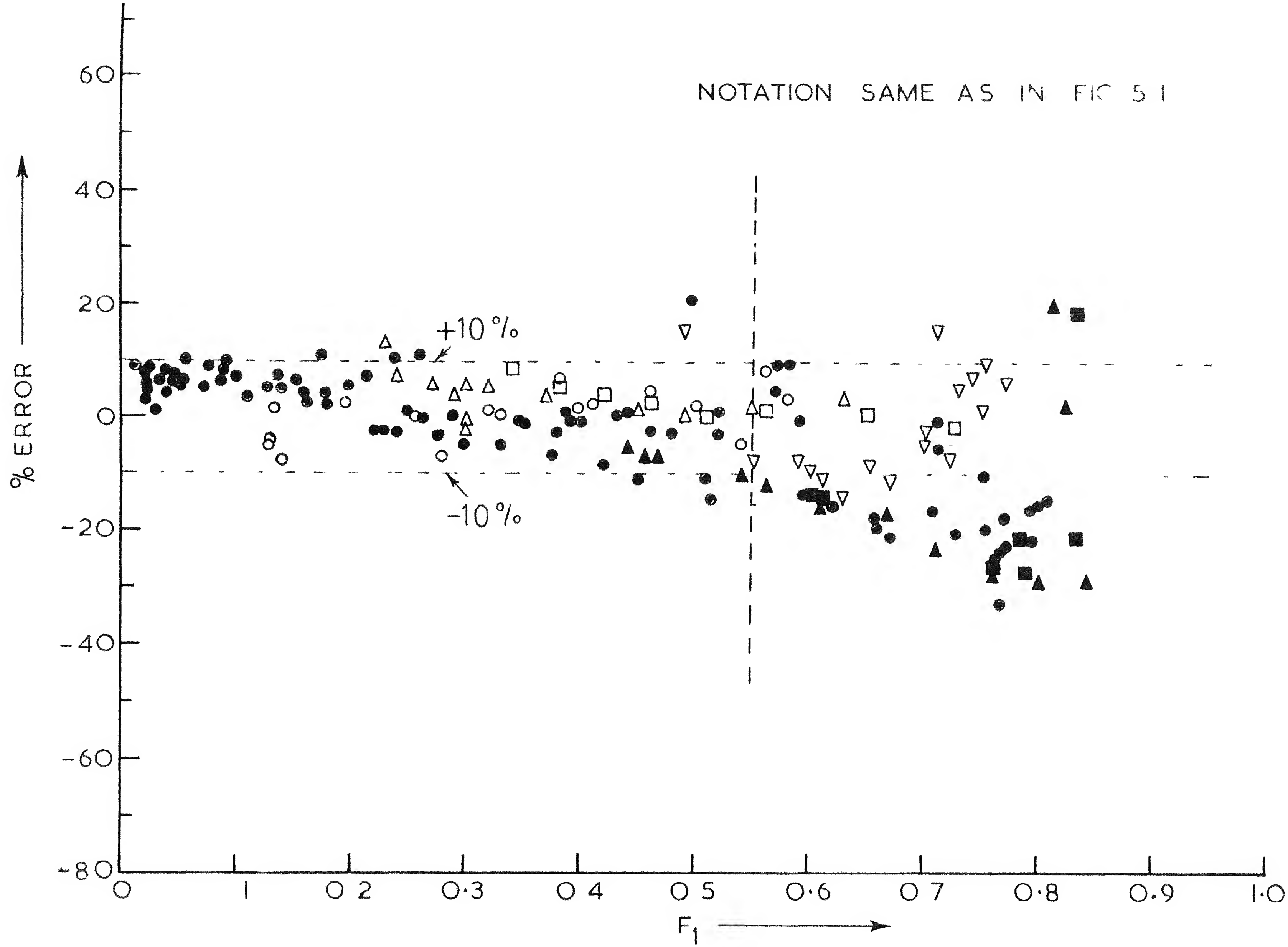


FIG. 5.3 PLOT OF % ERROR Vs F_1

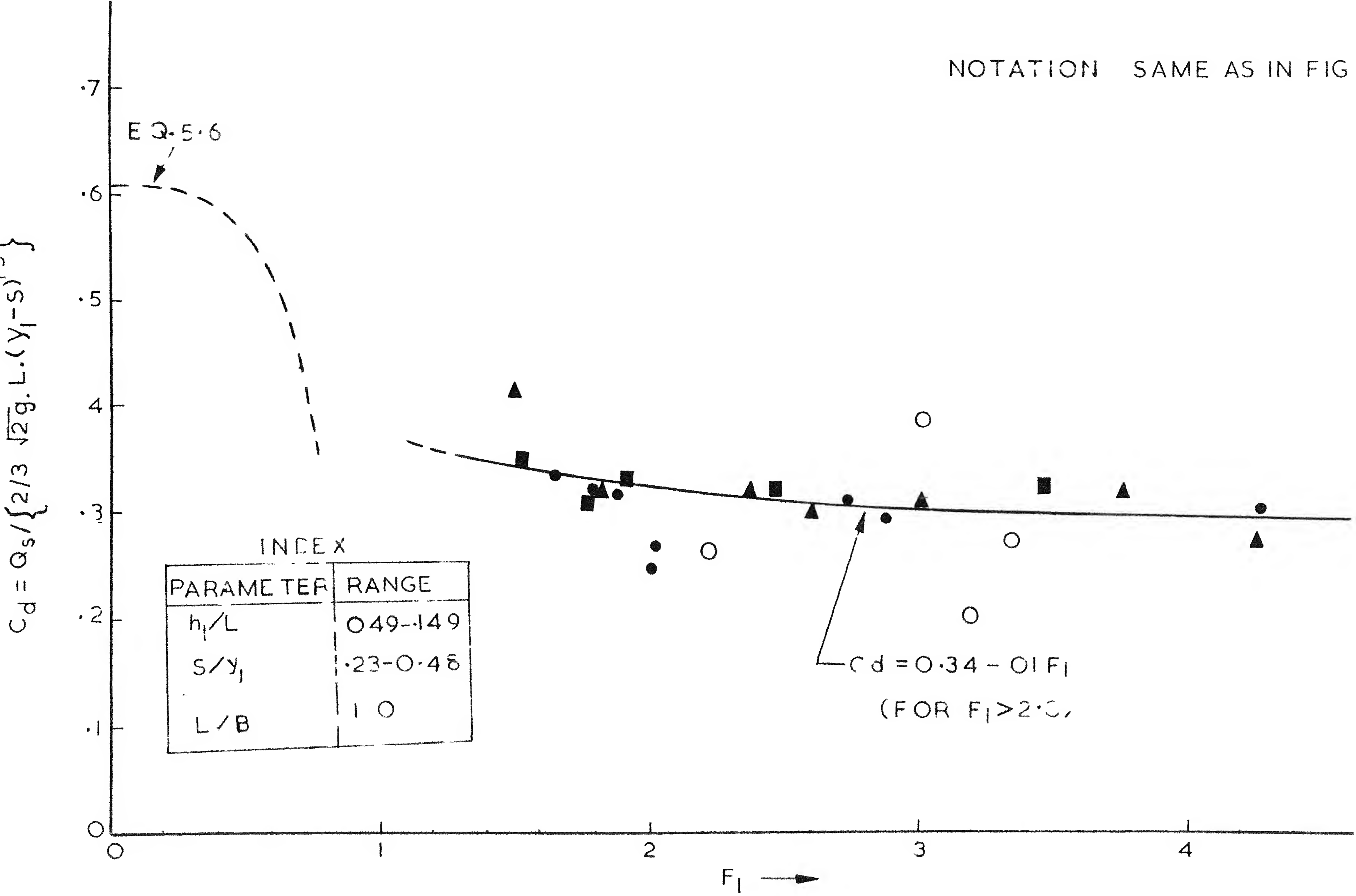


FIG. 5·4 VARIATION OF C_d WITH F_1 IN SUPERCRITICAL FLOW

follow the line given by equation (5.10). The scatter may be attributed to the experimental errors as in these experiments the available depth over the weir was very small (of the order of 2 cms.).

It is noted that the range of parameters L/B, h_1/L and s/y_1 in the supercritical flow experiments is very small to conclude any effect of these parameters on the equation (5.10). Further experimental evidence with wide range of the various parameters is needed.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

An extensive experimental study has been made on the behaviour of side-weirs in subcritical and supercritical flows. An analytical expression for the variation of De Marchi coefficient of discharge C_M has been derived. Based on the analytical as well as experimental results, the following conclusions are drawn:

1. The various empirical formulae for the determination of discharge over side-weirs are unsuitable for general use as they give varying degrees of error depending upon flow conditions; viz, Froude number.

2. De Marchi equation (2.12), which is based on logical premises could be used for discharge determination if proper discharge coefficient (C_M) is selected.

3. The De Marchi discharge coefficient (C_M) has been found to be function of only the upstream Froude number (F_1). The other parameters representing the geometry of the flow are found to have no effect on C_M, within the range used in the present investigation.

(i) Subcritical Flow:

Variation of discharge coefficient C_M is given by the equation

$$C_M = 0.611 \sqrt{1 - \frac{3F_1^2}{F_1^2+2}} \qquad \text{... Eqn. (4.11)}$$

(ii) Supercritical Flow:

Variation of discharge coefficient C_M is given by the equation

$$C_M = 0.36 - 0.08\ F_1 \qquad \text{...Eqn. (4.12)}$$

4. The solution of De Marchi equation (2.12) for design purposes involves tedious trial and error procedure.

An approximate formula (5.9) based on statistical correlations:

$$Q_s = C_d \cdot \frac{2}{3} \sqrt{2g} \cdot C\ (y_1 - s)^{3/2} \qquad (5.9)$$

where

$$C_d = \left(0.611 \sqrt{1 - \frac{3F_1^2}{F_1^2+2}}\right) \times \left(\frac{h_1}{L}\right)^{.06}$$

$$h_1 = y_1 - s$$

could be used in subcritical flow for Froude numbers less than .5 with an accuracy of $\pm$ 10%.

6.2 Recommendations:

1. The prediction of water surface profile in spatially varied flow along a side-weir by De Marchi equation needs to be verified by careful experimentation.

2. The exploratory studies on the hydraulic jump occuring along the side-weir (indicated in Appendix C) look promising. Further detailed studies are needed for the better understanding of the phenomenon.

3. The details of the separation zones occuring in downstream of side-weirs need to be studied.

4. Equation (5.9) for the determination of discharge Q_s over a side-weir looks promising because of its simplicity. However, experiments covering wide range of different parameters (i.e. h_1/L L/B and s/y_1) are needed to evaluate the effects of these parameters on coefficient C_d.

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APPENDIX A

EXPERIMENTAL DATA

APPENDIX A

EXPERIMENTAL DATA

A-1 FLUME A

Seri-	Ex. No.	L (cms)	B (cms)	s (cms)	y_1 (cms)	y_2 (cms)	Q_s (litres/sec.)	Q_1 (Litres/sec.)
AA	1	10.180	61.277	50.902	65.623	65.562	10.024	33.913
	2	10.180	61.277	50.902	58.552	58.308	3.771	34.836
	3	10.180	61.277	50.902	53.188	53.188	0.654	34.264
	4	10.180	61.277	50.902	62.514	62.393	7.007	19.253
AB	5	10.180	61.277	50.902	57.729	57.638	3.191	18.935
	6	10.180	61.277	50.902	54.376	54.285	1.196	18.901
	7	20.330	61.277	50.902	58.979	58.918	8.267	18.707
	8	20.330	61.277	50.902	53.889	53.736	1.954	18.218
	9	20.330	61.277	50.902	58.918	58.918	8.235	28.554
	10	20.330	61.277	50.902	58.979	58.979	8.283	16.423
	11	20.330	61.277	50.902	53.919	53.858	1.948	42.105
	12	20.330	61.277	50.902	53.889	53.889	1.961	28.542
	13	20.330	61.277	50.902	53.950	53.980	1.995	57.928
	14	20.330	61.277	50.902	53.919	53.919	1.941	70.007
	15	20.330	61.277	50.902	59.009	59.009	8.460	64.095
	16	20.330	61.277	50.902	56.937	56.937	5.475	40.497
	17	20.330	61.277	50.902	56.937	56.937	5.438	72.172
	18	20.330	61.277	50.902	59.893	59.863	9.811	45.324
	19	20.330	61.277	50.902	63.642	63.642	16.097	50.521
	20	20.330	61.277	38.222	40.264	40.173	1.126	48.715
	21	20.330	61.277	38.222	40.264	40.173	1.107	72.747
	22	20.330	61.277	38.222	40.264	40.173	1.146	17.433
	23	20.330	61.277	38.222	42.459	42.398	3.246	53.263
	24	20.330	61.277	38.222	45.019	44.988	6.403	59.739
	25	20.330	61.277	38.222	42.459	42.398	3.264	24.660
	26	20.330	61.277	38.222	42.459	42.428	3.218	72.916
	27	20.330	61.277	38.222	47.823	47.823	10.587	64.991
	28	20.330	61.277	38.222	50.993	50.993	15.861	72.342
	29	20.330	61.277	38.222	50.993	50.993	16.026	34.751
	30	20.330	61.277	38.222	54.590	54.620	23.089	38.600
	31	20.330	61.277	38.222	58.826	58.826	32.448	44.079
	32	20.544	61.277	25.481	26.853	26.853	0.613	46.726
	33	20.544	61.277	25.481	26.731	26.570	0.522	69.205
	34	20.544	61.277	25.481	28.346	28.316	1.752	68.375

A-1 FLUME A Contd...

eri-	Ex. No.	L (cms)	B (cms)	s (cms)	y_1 (cms)	y_2 (cms)	Q_s (Litres/ sec.)	Q_1 (litres/ sec.)
B	35	20.544	61.277	25.481	28.285	28.255	1.809	44.459
	36	20.544	61.277	25.481	30.053	30.084	3.712	44.278
	37	20.544	61.277	25.481	29.962	30.023	3.460	67.575
	38	20.544	61.277	25.481	31.791	31.852	5.779	68.290
	39	20.544	61.277	25.481	31.852	31.882	5.987	43.544
	40	20.544	61.277	25.481	33.924	33.985	9.137	44.009
	41	20.544	61.277	25.481	33.955	34.046	9.003	67.759
	42	20.544	61.277	25.481	36.393	36.515	13.045	68.038
	43	20.544	61.277	25.481	39,197	39.319	18.109	67.709
	44	20.544	61.277	25.481	42.459	42.611	24.645	69.746
	45	20.452	61.277	10.119	10.942	10.942	0.225	34.425
	46	20.452	61.277	10.119	11.918	11.918	0.886	34.130
	47.	20.452	61.277	10.119	12.649	12.649	1.221	49.263
	48	20.452	61.277	10.119	12.649	12.649	1.492	33.475
	49	20.452	61.277	10.119	13.503	13.442	1.752	55.475
	50	20.452	61.277	10.119	13.503	13.503	2.192	45.821
	51	20.452	61.277	10.119	14.508	14.508	2.595	60.486
	52	20.452	61.277	10.119	14.417	14.417	3.057	46.901
	53	20.452	61.277	10,119	15.575	15.636	4.531	46.382
	54	20.452	61.277	10.119	15.575	15.636	4.047	61.129
	55	20.452	61.277	10.119	15.545	15.636	3.761	70.109
	56	20.452	61.277	10.119	16.886	17.069	5.728	69.359
	57	20.452	61.277	10.119	16.947	17.160	6.106	61.494
	58	20.452	61,277	10.119	16.855	16.855	6.321	45.950
	59	20,452	61.277	10.119	20.269	20.269	11.787	45.729
	60	20,452	61.277	10.119	20.269	20.269	11.437	61.315
	61	20.452	61.277	10.119	25.298	25.298	20.950	61.311
	62	20.452	61.277	10.119	33.680	33.680	39.832	60.809
	63	20.452	61.277	10.119	33.680	33.680	39.791	70.809
	64	20.452	61.277	10.119	25.298	25.298	20.674	70.645
	65	20.452	61.277	10.119	20.269	20.269	11.016	70.595
	66	20.452	61.277	0.000	10.241	10.942	9.583	44.492
	67	20.452	61.277	0.000	10.241	10.729	11.417	32.506
	68	20.452	61.277	0.000	17.160	17.496	25.540	45.587
	69	20.452	61.277	0.000	21.031	21.184	34.872	45.238
	70	20.452	61.277	0.000	8.230	8.992	7.706	29.724
	71	20.452	61.277	0.000	16.551	16.703	24.859	29.291
	72	20.452	61.277	0.000	11.186	11.460	12.085	55.159
	73	20.452	61.277	0.000	19.964	20.086	32.197	52.926
	74	20.452	61.277	0.000	16.032	16.551	22.738	52.762

A-1 FLUME A Contd...

Series	Ex. No.	L (cms)	B (cms)	s (cms)	y_1 (cms)	y_2 (cms)	Q_s (Litres/sec.)	Q_1 (Litres/sec.)
AB	75	20.452	61.277	0.000	24.079	24.293	42.565	51.821
	76	20.452	61.277	0.000	12.283	12.253	10.294	58.974
	77	20.452	61.277	0.000	10.058	10.881	9.205	46.840
	78	20.452	61.277	0.000	10.485	11.034	9.462	50.275
	79	20.452	61.277	0.000	4.298	4.481	1.758	46.728
	80	20.452	61.277	0.000	5.639	5.761	2.668	48.032
	81	20.452	61.277	0.000	10.455	11.034	9.393	48.900
	82	20.452	61.277	0.000	20.361	20.330	33.135	48.508
	83	20.452	61.277	0.000	20.361	20.330	33.135	48.508
	84	20.452	61.277	0.000	16.581	17.008	24.220	48.622
	85	20.452	61.277	0.000	12.070	12.040	10.131	63.854
	86	20.452	61.277	0.000	17.800	17.770	26.740	65.007
	87	20.452	61.277	0.000	6.157	6.309	5.563	11.125
	88	20.452	61.277	0.000	11.278	11.735	9.864	57.724
	89	20.452	61.277	0.000	15.270	16.185	21.284	58.762
	90	20.452	61.277	0.000	22.494	22.951	38.267	57.226
	91	20.452	61.277	0.000	6.492	6.523	3.237	56.717
	92	20.452	61.277	0.000	5.456	5.182	2.362	70.373
	93	20.452	61.277	0.000	4.054	3.993	1.568	66.985
	94	20.452	61.277	0.000	6.858	6.401	3.022	70.977
	95	20.452	61.277	0.000	7.529	7.468	4.120	71.185
	96	20.452	61.277	0.000	7.315	6.858	4.141	62.847
	97	20.452	61,277	0.000	6.340	5.974	2.454	61.724
	98	20.452	61.277	0.000	13.045	12.984	11.092	72.231
	99	20.452	61.277	0.000	14.691	15.118	18.008	72.755
	100	20.452	61.277	0.000	13.533	14.173	16.192	56.841
	101	20.452	61.277	0.000	11.552	11.704	9.723	56.682
	102	20.452	61.277	0.000	10.668	10.607	8.852	47.677
	103	20.452	61.277	0.000	11.674	12.253	12.920	47.754
	104	20.452	61.277	0.000	12.131	13.289	13.638	53.429
	105	20.452	61.277	0.000	10.820	10.881	9.290	52.705
	106	20.452	61.277	0.000	12.924	12.588	10.624	71.815
	107	20.452	61.277	0.000	13.899	14.508	15.052	72.336
AC	108	38.130	61.277	0.000	8.108	9.388	11.417	37.301
	109	38.130	61.277	0.000	9.510	10.333	17.957	37.576
	110	38.130	61.277	0.000	11.003	11.674	23.680	37.839
	111	38.130	61.277	0.000	12.101	12.710	28.139	37.932
	112	38.130	61.277	0.000	10.942	9.357	12.837	57.328
	113	38.130	61.277	0.000	11.278	12.558	20.839	58.200
	114	38.130	61.277	0.000	11.887	13.411	24.890	56.130
	115	38.130	61.277	0.000	13.228	14.356	30.680	56.470
	116	38.130	61.277	0.000	15.972	16.551	42.354	56.165
	117	38.130	61.277	0.000	12.710	9.418	14.071	70.502

A-1 FLUME A Contd...

Seri-es	Ex. No.	L (cms)	B (cms)	s (cms)	y_1 (cms)	y_2 (cms)	Q_s (Litres/sec.)	Q_1 (Litres/sec.)
AC	118	38.130	61.277	0.000	13.472	14.996	29.005	72.122
	119	38.130	61.277	0.000	14.661	16.063	34.275	71.363
	120	38.130	61.277	0.000	16.307	17.343	41.976	71.083
	121	38.130	61.277	0.000	19.050	19.964	51.258	70.049
	122	38.130	61.277	0.000	8.473	8.626	12.045	70.495
	123	38.130	61.277	0.000	7.315	7.041	7.448	69.271
	124	38.130	61.277	0.000	6.157	6.157	5.753	69.170
	125	38.130	61.277	0.000	5.243	5.608	4.443	68.990
	126	38.130	61.277	0.000	4.298	4.968	2.953	72.707
	127	38.130	61.277	0.000	4.115	4.694	3.191	60.173
	128	38.130	61.277	0.000	5.33[illegible]	5.730	4.432	61.414
AD	129	61.021	61.277	0.000	9.083	5.883	13.616	43.606
	130	61.021	61.277	0.000	9.114	9.906	20.591	44.034
	131	61.021	61.277	0.000	9.601	10.820	28.504	43.579
	132	61.021	61.277	0.000	11.034	11.918	38.148	42.791
	133	61.021	61.277	0.000	14.143	15.210	54.551	61.316
	134	61.021	61.277	0.000	6.980	6.279	12.105	53.997
	135	61.021	61.277	0.000	4.999	5.273	6.935	52.475
	136	61.021	61.277	0.000	3.871	4.298	4.815	50.486
	137	61.021	61.277	0.000	11.430	6.767	16.503	60.605
	138	61.021	61.277	0.000	11.796	10.333	36.430	61.228
	139	61.021	61,277	0.000	13.746	9.449	20.318	77.401
	140	61.021	61.277	0.000	13.929	15.728	43.202	78.186
	141	61.021	61.277	0.000	7.529	7.163	12.920	75.909
A[illegible]	142	38.100	61.277	21.519	33.162	33.680	25.508	99.650
	143	38.100	61.277	21.519	37.247	37.795	38.704	100.001
	144	38.100	61.277	21.519	30.480	30.876	17.605	89.938
	145	38.100	61.277	21.519	28.743	29.047	12.878	89.094
	146	38.100	61.277	21.519	28.103	28.438	11.650	51.198
	147	38.100	61.277	21.519	31.852	32.095	22.104	51.480
	148	38.100	61.277	8.016	15.423	16.459	11.886	86.381
	149	38.100	61.277	8.016	19.568	20.726	25.384	86,733
	150	38.100	61.277	8.016	22.128	26.243	33.390	84.956
	151	38.100	61.277	8.016	17.831	18.166	20.729	41.512
	152	38.100	61.277	8.016	20.513	20.818	29.648	41.298
	153	38.100	61.277	8.016	13.533	13.716	8.219	40.847
	154	38.100	61.277	8.016	12.924	13.442	7.300	50.715
	155.	38.100	61.277	8.016	15.850	16.764	13.531	86.444
	156	38.100	61.277	8.016	18.928	19.964	23.030	85.647

A-1 FLUME A

Series	Ex. No.	L (cms)	B (cms)	s (cms)	y_1 (cms)	y_2 (cms)	Q_s (Litres/sec.)	Q_1 (Litres/sec.)
AF	157	60.960	61.277	7.864	12.497	14.143	10.276	67.812
	158	60.960	61.277	7.864	15.362	16.886	23.532	69.468
	159	60.960	61.277	7.864	20.696	21.610	44.491	64.320
	160	60.960	61.277	7.864	21.214	23.409	52.375	102.765
	161	60.960	61.277	7.864	17.221	20.300	31.169	103.792
	162	60.960	61.277	2.743	6.553	5.883	3.527	71.593
	163	60.960	61.277	2.743	5.304	5.060	2.867	70.655
	164	60.960	61.277	2.743	4.877	4.846	1.527	69.147
	165	60.960	61.277	2.743	5.456	5.608	1.515	76.305
AG	DATA OF THE HYDRAULIC JUMP							
	166	84.000	61.227	0.000	0.155	0.342	1.35	1.728
	167	84.00	61.277	0.000	0.115	0.403	1.267	1.645
	168	84.000	61.277	0.000	0.099	0.475	1.120	1.565
	169	84.000	61.277	0.000	0.135	0.608	2.065	2.417
	170	84.000	61.277	0.000	0.133	0.507	2.27	2.662
	171	84.000	61.277	0.000	0.142	0.710	2.27	2.605
	172	84.000	61.277	0.000	0.192	0.461	2.27	2.567
	173	60.96	61.277	2.74	0.190	0.700	0.147	2.82

A-2 FLUME B

Series	Ex. No.	L (cms)	B (cms)	s (cms)	y_1 (cms)	y_2 (cms)	Q_s (Litres/sec.)	Q_1 (Litres/sec.)
BA	1	5.100	24.800	0.000	3.700	3.550	0.578	2.196
	2	5.100	24.800	0.000	3.660	3.470	0.466	3.049
	3	5.100	24.800	0.000	3.660	3.300	0.528	2.518
	4	5.100	24.800	0.000	3.650	3.400	0.621	1.393
	5	5.100	24.800	0.000	3.640	3.400	0.633	0.722
	6	5.100	24.800	0.000	3.640	3.400	0.618	1.052
	7	5.100	24.800	0.000	3.640	3.280	0.597	1.775
	8	5.100	24.800	0.000	3.630	3.270	0.555	2.206
	9	5.100	24.800	0.000	4.140	3.760	0.623	3.311
	10	5.100	24.800	0.000	3.660	3.000	0.466	3.056
	11	5.100	24.800	0.000	3.660	3.000	0.528	2.518
	12	5.100	24.800	0.000	3.650	3.000	0.621	1.393
	13	5.100	24.800	0.000	3.700	3.100	0.578	2.196
	14	5.100	24.800	0.000	8.280	7.600	2.213	2.391
	15	5.100	24.800	0.000	6.810	6.000	1.656	1.801
	16	5.100	24.800	0.000	3.020	2.900	0.452	1.321
	17	5.100	24.800	0.000	2.730	2.600	0.360	1.333
	18	5.100	24.800	0.000	6.890	6.000	1.647	4.057
	19	5.100	24.800	0.000	4.530	4.190	0.732	4.071
	20	5.100	24.800	0.000	12.400	11.500	4.051	4.906

contd..

A-2 FLUME B

Series	Ex. No.	L (cms)	B (cms)	s (cms)	y_1 (cms)	y_2 (cms)	Q_s (Litres/sec.)	Q_1 (Litres/sec.)
BB	21	9.960	24.800	0.000	3.680	3.470	1.201	1.516
	22	9.960	24.800	0.000	3.680	3.500	0.985	3.175
	23	9.960	24.800	0.000	3.670	3.380	1.025	3.022
	24	9.960	24.800	0.000	3.680	3.510	1.101	2.723
	25	9.960	24.800	0.000	3.670	3.480	1.126	2.488
	26	9.960	24.800	0.000	3.670	3.500	1.147	2.264
	27	9.960	24.800	0.000	3.690	3.630	1.174	2.033
	28	9.960	24.800	0.000	3.670	3.360	1.178	1.769
	29	9.960	24.800	0.000	3.680	3.340	1.193	1.647
	30	9.960	24.800	0.000	3.680	3.300	1.202	1.316
	31	9.960	24.800	0.000	5.040	4.710	1.921	2.574
	32	9.960	24.800	0.000	6.150	5.930	2.655	3.606
	33	9.960	24.800	0.000	6.890	6.680	3.169	4.317
	34	9.960	24.800	0.000	4.800	4.470	1.349	5.156
	35	9.960	24.800	0.000	2.060	1.800	0.434	0.793
	36	9.960	24.800	0.000	2.670	2.260	0.716	0.786
BC	37	15.000	24.800	0.000	3.670	3.320	1.738	1.850
	38	15.000	24.800	0.000	3.670	3.330	1.731	2.101
	39	15.000	24.800	0.000	3.670	3.380	1.717	2.322
	40	15.000	24.800	0.000	3.680	3.430	1.697	2.553
	41	15.000	24.800	0.000	3.700	3.480	1.681	2.851
	42	15.000	24.800	0.000	3.660	3.510	1.587	3.041
	43	15.000	24.800	0.000	3.680	3.570	1.419	3.574
	44	15.000	24.800	0.000	4.290	4.450	1.609	5.004
	45	15.000	17.550	0.000	3.950	3.900	1.890	2.730
	46	15.000	17.550	0.000	3.440	3.300	1.1516	2.138
	47	15.000	17.550	0.000	2.820	2.530	1.075	1.459
	48	15.000	17.550	0.000	1.940	1.590	0.583	0.737
	49	15.000	17.550	0.000	3.940	3.800	1.218	3.086
	50	15.000	17.550	0.000	4.220	4.260	1.351	3.564
	51	15.000	17.550	0.000	3.750	3.640	1.218	3.012
	52	15.000	17.550	0.000	3.400	3.290	1.390	2.252
	53	15.000	17.550	0.000	3.790	3.600	1.819	2.253
	54	15.000	17.550	0.000	3.530	3.220	1.559	2.185
	55	15.000	17.550	0.000	3.940	3.840	1.859	2.653
	56	15.000	17.550	0.000	3.790	3.710	1.417	2.848
	57	15.000	17.550	0.000	3.850	3.820	1.674	2.777
	58	15.000	17.550	0.000	3.860	3.820	1.473	2.954
	59	15.000	17.550	0.000	4.050	4.060	1.580	3.238
	60	15.000	17.550	0.000	4.120	4.050	1.384	3.398
	61	15.000	17.550	0.000	4.230	4.040	1.390	3.569
	62	15.000	17,550	0.000	4.640	4.770	1.533	4.225

NON DIMENSIONAL PARAMETERS

A-3 FLUME A

Series	Exp. No.	h_1/L	L/B	s/y_1	F_1
AA	1	1.4461	0.1661	0.7757	0.0332
	2	0.7515	0.1661	0.8693	0.0405
	3	0.2246	0.1661	0.9570	0.0460
	4	1.1407	0.1661	0.8142	0.0203
AB	5	0.6707	0.1661	0.8817	0.0225
	6	0.3413	0.1661	0.9361	0.0246
	7	0.3973	0.3318	0.8630	0.0215
	8	0.1469	0.3318	0.9446	0.0240
	9	0.3943	0.3318	0.8639	0.0329
	10	0.3973	0.3318	0.8630	0.0189
	11	0.1484	0.3318	0.9440	0.0554
	12	0.1469	0.3318	0.9446	0.0376
	13	0.1499	0.3318	0.9435	0.0762
	14	0.1484	0.3318	0.9440	0.0921
	15	0.3988	0.3318	0.8626	0.0737
	16	0.2969	0.3318	0.8940	0.0491
	17	0.2969	0.3318	0.8940	0.0875
	18	0.4423	0.3318	0.8499	0.0509
	19	0.6267	0.3318	0.7998	0.0518
	20	0.1004	0.3318	0.9493	0.0993
	21	0.1004	0.3318	0.9493	0.1484
	22	0.1004	0.3318	0.9493	0.0356
	23	0.2084	0.3318	0.9002	0.1003
	24	0.3343	0.3318	0.8490	0.1030
	25	0.2084	0.3318	0.9002	0.0464
	26	0.2084	0.3318	0.9002	0.1373
	27	0.4723	0.3318	0.7992	0.1024
	28	0.6282	0.3318	0.7496	0.1035
	29	0.6282	0.3318	0.7496	0.0497
	30	0.8051	0.3318	0.7002	0.0499
	31	1.0135	0.3318	0.6497	0.0509
	32	0.0668	0.3353	0.9489	0.1750
	33	0.0608	0.3353	0.9532	0.2609
	34	0.1395	0.3353	0.8989	0.2361
	35	0.1365	0.3353	0.9009	0.1540
	36	0.2226	0.3353	0.8479	0.1100
	37	0.2181	0.3353	0.8505	0.2147
	38	0.3071	0.3353	0.8015	0.1985
	39	0.3101	0.3353	0.8000	0.1262
	40	0.4110	0.3353	0.7511	0.1161

contd...

A-3 FLUME A

Series	Exp. No.	h_1/L	L/B	s/y_1	F_1
AB	41	0.4125	0.3353	0.7504	0.1784
	42	0.5312	0.3353	0.7002	0.1615
	43	0.6677	0.3353	0.6501	0.1438
	44	0.8264	0.3353	0.6001	0.1314
	45	0.0402	0.3338	0.9248	0.4956
	46	0.0879	0.3338	0.8491	0.4322
	47	0.1237	0.3338	0.8000	0.5706
	48	0.1237	0.3338	0.8000	0.3877
	49	0.1654	0.3338	0.7494	0.5826
	50	0.1654	0.3338	0.7494	0.4812
	51	0.2146	0.3338	0.6975	0.5703
	52	0.2101	0.3338	0.7019	0.4464
	53	0.2668	0.3338	0.6497	0.3932
	54	0.2668	0.3338	0.6497	0.5182
	55	0.2653	0.3338	0.6510	0.5960
	56	0.3308	0.3338	0.5993	0.5208
	57	0.3338	0.3338	0.5971	0.4593
	58	0.3294	0.3338	0.6004	0.3460
	59	0.4963	0.3338	0.4992	0.2611
	60	0.4963	0.3338	0.4992	0.3501
	61	0.7422	0.3338	0.4000	0.2511
	62	1.1520	0.3338	0.3005	0.1621
	63	1.1520	0.3338	0.3005	0.1881
	64	0.7422	0.3338	0.4000	0.2893
	65	0.4963	0.3338	0.4992	0.4031
	66	0.5007	0.3338	0.0000	0.7073
	67	0.5007	0.3338	0.0000	0.5168
	68	0.8390	0.3338	0.0000	0.3341
	69	1.0283	0.3338	0.0000	0.2444
	70	0.4024	0.3338	0.0000	0.6560
	71	0.8092	0.3338	0.0000	0.2267
	72	0.5469	0.3338	0.0000	0.7682
	73	0.9762	0.3338	0.0000	0.3091
	74	0.7839	0.3338	0.0000	0.4282
	75	1.1773	0.3338	0.0000	0.2285
	76	0.6006	0.3338	0.0000	0.7138
	77	0.4918	0.3338	0.0000	0.7651
	78	0.5127	0.3338	0.0000	0.7715
	79	0.2101	0.3338	0.0000	2.7327
	80	0.2757	0.3338	0.0000	1.8690
	81	0.5112	0.3338	0.0000	0.7537
	82.	0.9955	0.3338	0.0000	0.2751
	83	0.9955	0.3338	0.0000	0.2751
	84	0.8107	0.3338	0.0000	0.3752

contd...

A-3 FLUME A

Series	Exp. No.	h_1/L	L/B	s/y_1	F_1
AB	85	0.5902	0.3338	0.0000	0.7934
	86	0.8703	0.3338	0.0000	0.4510
	87	0.3010	0.3338	0.0000	0.3794
	88	0.5514	0.3338	0.0000	0.7942
	89	0.7466	0.3338	0.0000	0.5131
	90	1.0999	0.3338	0.0000	0.2795
	91	0.3174	0.3338	0.0000	1.7865
	92	0.2668	0.3338	0.0000	2.8772
	93	0.1982	0.3338	0.0000	4.2761
	94	0.3352	0.3338	0.0000	0.0592
	95	0.3681	0.3338	0.0000	1.7955
	96	0.3577	0.3338	0.0000	1.6551
	97	0.3100	0.3338	0.0000	2.0147
	98	0.6379	0.3338	0.0000	0.7987
	99	0.7183	0.3338	0.0000	0.6732
	100	0.6617	0.3338	0.0000	0.5949
	101	0.5648	0.3338	0.0000	0.7522
	102	0.5216	0.3338	0.0000	0.7129
	103	0.5708	0.3338	0.0000	0.6238
	104	0.5931	0.3338	0.0000	0.6589
	105	0.5291	0.3338	0.0000	0.7715
	106	0.6319	0.3338	0.0000	0.8054
	107	0.6796	0.3338	0.0000	0.7274
AC	108	0.2126	0.6223	0.0000	0.8419
	109	0.2494	0.6223	0.0000	0.6676
	110	0.2886	0.6223	0.0000	0.5402
	111	0.3173	0.6223	0.0000	0.4695
	112	0.2870	0.6223	0.0000	0.8252
	113	0.2958	0.6223	0.0000	0.8007
	114	0.3118	0.6223	0.0000	0.7136
	115	0.3469	0.6223	0.0000	0.6116
	116	0.4189	0.6223	0.0000	0.4585
	117	0.3333	0.6223	0.0000	0.8107
	118	0.3533	0.6223	0.0000	0.7599
	119	0.3845	0.6223	0.0000	0.6624
	120	0.4277	0.6223	0.0000	0.5624
	121	0.4996	0.6223	0.0000	0.4390
	122	0.2222	0.6223	0.0000	1.4892
	123	0.1918	0.6223	0.0000	1.8242
	124	0.1615	0.6223	0.0000	0.3591
	125	0.1375	0.6223	0.0000	0.9946
	126	0.1127	0.6223	0.0000	4.2520
	127	0.1079	0.6223	0.0000	3.7562

contd...

A-3 FLUME A

Series	Exp. No.	h_1/L	L/B	s/y_1	F_1
AC	128	0.1399	0.6223	0.0000	2.5975
AD	129	0.1489	0.9958	0.0000	0.8300
	130	0.1494	0.9958	0.0000	0.8339
	131	0.1573	0.9958	0.0000	0.7632
	132	0.1808	0.9958	0.0000	0.6083
	133	0.2318	0.9958	0.0000	0.6007
	134	0.1144	0.9958	0.0000	1.5257
	135	0.0819	0.9958	0.0000	2.4465
	136	0.0634	0.9958	0.0000	3.4540
	137	0.1873	0.9958	0.0000	0.8172
	138	0.1933	0.9958	0.0000	0.7875
	139	0.2253	0.9958	0.0000	0.7913
	140	0.2283	0.9958	0.0000	0.7836
	141	0.1234	0.9958	0.0000	1.9147
AE	142	0.3056	0.6218	0.6489	0.2719
	143	0.4128	0.6218	0.5777	0.2292
	144	0.2352	0.6218	0.7060	0.2785
	145	0.1896	0.6218	0.7487	0.3013
	146	0.1728	0.6218	0.7657	0.1791
	147	0.2712	0.6218	0.6756	0.1492
	148	0.1944	0.6218	0.6198	0.7431
	149	0.3032	0.6218	0.4097	0.5221
	150	0.3704	0.6218	0.3623	0.4252
	151	0.2576	0.6218	0.4496	0.2873
	152	0.3280	0.6218	0.3808	0.2316
	153	0.1448	0.6218	0.5923	0.4275
	154	0.1288	0.6218	0.6203	0.5688
	155	0.2056	0.6218	0.5058	0.7138
	156	0.2864	0.6218	0.4235	0.5419
AF	157	0.0760	0.9948	0.6293	0.7998
	158	0.1230	0.9948	0.5119	0.6012
	159	0.2105	0.9948	0.3800	0.3560
	160	0.2190	0.9948	0.3707	0.5480
	161	0.1535	0.9948	0.4566	0.7567
	162	0.0625	0.9948	0.4186	2.2236
	163	0.0420	0.9948	0.5172	3.0142
	164	0.0350	0.9948	0.5625	3.3454
	165	0.0445	0.9948	0.5028	3.1198

A-3 FLUME A

Series	Exp. No.	h_1/L	L/B	s/y_1	F_1
AG	DATA OF THE HYDRAULIC JUMP				
	166	0.077	1.375	0.0	2.48
	167	0.057	1.375	0.0	3.70
	168	0.048	1.375	0.0	4.43
	169	0.067	1.375	0.0	4.27
	170	0.066	1.375	0.0	4.82
	171	0.071	1.375	0.0	4.23
	172	0.096	1.375	0.0	2.68
	173	0.05	1.00	0.47	2.99

A-4 FLUME B

Series	Exp. No.	h_1/L	L/B	s/y_1	F_1
BA	1	0.7255	0.2056	0.0000	0.3973
	2	0.7176	0.2056	0.0000	0.5606
	3	0.7176	0.2056	0.0000	0.4630
	4	0.7157	0.2056	0.0000	0.2573
	5	0.7137	0.2056	0.0000	0.1339
	6	0.7137	0.2056	0.0000	0.1949
	7	0.7137	0.2056	0.0000	0.3290
	8	0.7118	0.2056	0.0000	0.4106
	9	0.8118	0.2056	0.0000	0.5060
	10	0.7176	0.2056	0.0000	0.5620
	11	0.7176	0.2056	0.0000	0.4630
	12	0.7157	0.2056	0.0000	0.2573
	13	0.7255	0.2056	0.0000	0.3973
	14	1.6235	0.2056	0.0000	0.1292
	15	1.2353	0.2056	0.0000	0.1305
	16	0.5922	0.2056	0.0000	0.3241
	17	0.5353	0.2056	0.0000	0.3806
	18	1.3510	0.2056	0.0000	0.2888
	19	0.8882	0.2056	0.0000	0.5436
	20	2.4314	0.2056	0.0000	0.1447
BB	21	9.3695	0.4016	0.0000	0.2765
	22	0.3695	0.4016	0.0000	0.5790
	23	0.3685	0.4016	0.0000	0.5534
	24	0.3695	0.4016	0.0000	0.4967
	25	0.3685	0.4016	0.0000	0.4556
	26	0.3685	0.4016	0.0000	0.4145
	27	0.3705	0.4016	0.0000	0.3692
	28	0.3685	0.4016	0.0000	0.3239
	29	0.3695	0.4016	0.0000	0.3004
	30	0.3695	0.4016	0.0000	0.2400
	31	0.5060	0.4016	0.0000	0.2929

contd..

A-4 FLUME B

Series	Exp. No.	h_1/L	L/B	s/y_1	F_1
IB	32	0.6175	0.4016	0.0000	0.3044
	33	0.6918	0.4016	0.0000	0.3073
	34	0.4819	0.4016	0.0000	0.6312
	35	0.2068	0.4016	0.0000	0.3453
	36	0.2681	0.4016	0.0000	0.2319
BC	37	0.2447	0.6048	0.0000	0.3387
	38	0.2447	0.6048	0.0000	0.3846
	39	0.2447	0.6048	0.0000	0.4251
	40	0.2453	0.6048	0.0000	0.4655
	41	0.2467	0.6048	0.0000	0.5157
	42	0.2440	0.6048	0.0000	0.5591
	43	0.2453	0.6048	0.0000	0.6518
	44	0.2860	0.6048	0.0000	0.7250
	45	0.2633	0.8547	0.0000	0.6326
	46	0.2293	0.8547	0.0000	0.6096
	47	0.1880	0.8547	0.0000	0.5607
	48	0.1293	0.8547	0.0000	0.4962
	49	0.2627	0.8547	0.0000	0.7179
	50	0.2813	0.8547	0.0000	0.7479
	51	0.2500	0.8547	0.0000	0.7547
	52	0.2267	0.8547	0.0000	0.6534
	53	0.2527	0.8547	0.0000	0.5556
	54	0.2353	0.8547	0.0000	0.5992
	55	0.2627	0.8547	0.0000	0.6172
	56	0.2527	0.8547	0.0000	0.7021
	57	0.2567	0.8547	0.0000	0.6688
	58	0.2573	0.8547	0.0000	0.7086
	59	0.2700	0.8547	0.0000	0.7228
	60	0.2747	0.8547	0.0000	0.7392
	61	0.2820	0.8547	0.0000	0.7462
	62	0.3093	0.8547	0.0000	0.7690

APPENDIX B

A DESIGN PROBLEM

APPENDIX B

In the field application of side-weir, two types of problems are met.

(a) Given upstream flow conditions (Q_1, y_1) the problem is to find length of the weir to spill over the required discharge Q_s. In this case the crest height may be decided upon a certain discharge (say Q_o) in the main channel above which the weir can start functioning. The height of the weir crest will be given by the normal depth corresponding to Q_o which may be calculated by Mannings formula; viz.

$$Q_o = \frac{1}{n} AR^{2/3} S_o^{1/2} \quad \text{in metric units} \quad (B-1)$$

(b) Given upstream flow conditions (Q_1, y_1), length and height of the weir crest (L, s) the problem is to find the discharge Q_s passing over the weir. The solution involves a tedious trial and error procedure.

The following example illustrates the procedures of solving both the types of problems. The problem has also been solved on computer for efficient solution and the corresponding programmes are given.

<u>Example</u>: A long channel of rectangular section 2m wide is laid on a slope of 0.001 and is lined with concrete, n = 0.014. Two side-weirs are to be installed, of equal length and on opposite

sides of the channel. They are to come in to operation when the flow reaches .6 m^3/sec and are to discharge .15 m^3/sec. when the upstream flow is .9 m^3/sec. Determine the length of the weirs, and the height of their crests above the channel bed. If the length is now made equal to 2.0 m, the crest height remaining the same what will be the discharge over the weirs for the same upstream flow of .9 m^3/sec.

Given $B = 2.0$ m; slope $S_o = .001$

$n = .014$, Q_b = the discharge in the channel when the weir starts operating = 0.6 m^3/sec.

Q_1 = Final upstream discharge = 0.9 m^3/sec.

Solution (Part A)

Given $Q_s = .15$ m^3/sec. To find L, the length of the weir

Section factor $Z = AR^{2/3} = \dfrac{nQ}{\sqrt{S_o}}$

$$= \frac{0.014 \times .6}{\sqrt{.001}} = .2655$$

$$B^{8/3} = 2^{8/3} = 6.35$$

$$AR^{2/3}/B^{8/3} = \frac{.2655}{6.35} = .0418$$

From the graph (ref. Chow V.T., 'Open channel Hydraulics' pp.130), the normal depth is given by $y_n/B = .165$, $y_n = .165 \text{x} 2 = .33$

So height of the weir crest, $s = .33$ m.

The normal depth corresponding to upstream flow ($.9 m^3/sec.$),

$$AR^{2/3} = \frac{nQ}{\sqrt{S_o}} = .0418 \times \frac{.9}{.6} = .0628$$

From the graph

$$y_1/B = .22$$

$$y_1 = .44 \ m$$

$$V_1 = \frac{Q_1}{Bxy_1} = \frac{.9}{2x.44} = 1.022 \ m/sec.$$

Specific energy $E = y_1 + V_1{}^2/2g$

$$= .44 + \frac{(1.022)^2}{2x9.81} = .4962 \ m$$

Upstream Froude number

$$F_1 = \frac{V_1}{\sqrt{gy_1}} = \frac{1.022}{\sqrt{9.81x.44}} = .492$$

De Marchi equation for two weirs placed opposite to each other is

$$\frac{L}{B} = \frac{3}{4\ C_M}(\phi_2 - \phi_1)$$

$$s/y_1 = .33/.44 = .75, \ F_1 = .492$$

From the Fig. B-1,

$$\phi_1 = -1.85$$

$$\phi_2 = Q_1 - Q_s = .9 - .15 = .75 \ m^3/sec.$$

$$E = y_2 + \frac{V_2{}^2}{2g} = y_2 + \frac{Q_2{}^2}{B^2 y_2{}^2 . 2g}$$

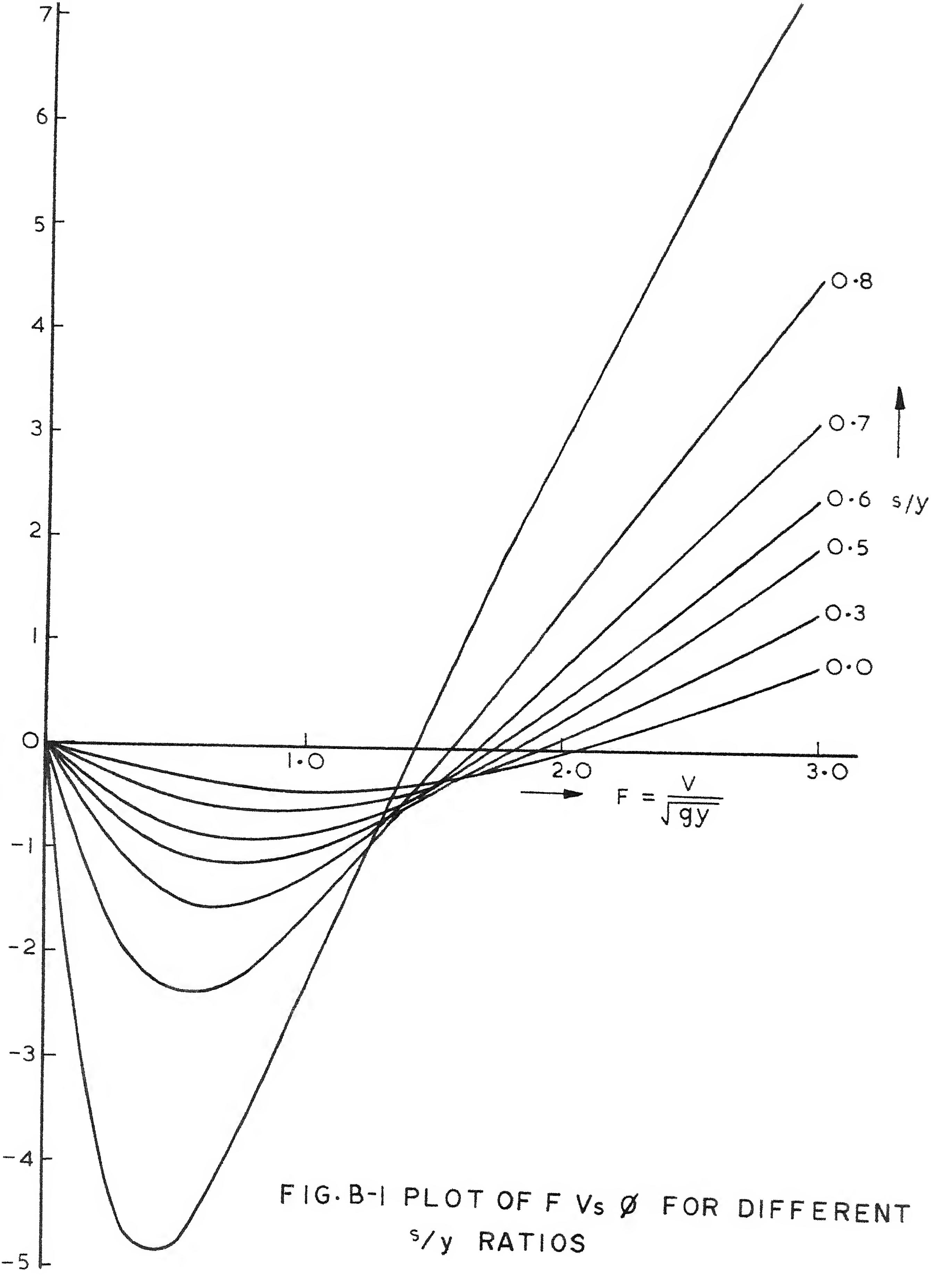

FIG. B-1 PLOT OF F Vs Ø FOR DIFFERENT s/y RATIOS

$$.4962 = y_2 + \frac{.75x.75}{y_2^2 x\ 8x9.81}$$

or

$$.4962 = y_2 + \frac{.00717}{y_2^2}$$

Solving by trial and error method

$$y_2 = .4627\ m$$

$$s/y_2 = .33/.4627 = .713$$

$$F_2 = \sqrt{2(\frac{E}{y_2} - 1)} = \sqrt{2(\frac{.4962}{.4627} - 1)}$$

$$= .38$$

From Fig. B-1, for F_2 = .38 and, s/y_2 = .713

$$\emptyset_2 = -1.35$$

$$\emptyset_2 - \emptyset_1 = .50$$

$$C_M = .611 \sqrt{1 - \frac{3F_1^2}{F_1^2+2}} \qquad \text{(Ref. Eq. 4.11)}$$

$$= .50$$

$$\frac{L}{B} = \frac{3}{4x.5}(.50) = .75$$

$L = 1.5$ m Answer

<u>Solution Part II</u>

Given L = 2.0 m i.e. L/B = 1.0

To find Q_s,

Substituting in De Marchi equation

$$1.0 = \frac{3}{4x.5} (\phi_2 - (-1.85))$$

$$\phi_2 = -1.85 + .667 = -1.183$$

A trial and error method has to be adopted to solve for y_2 and hence for Q_s.

Step I assuming $y_2 = .48$

$$s/y_2 = \frac{.33}{.48} = .687, \ \frac{E}{y_2} = \frac{.4962}{.48} = 1.033$$

$$F_2 = \sqrt{2(1.033-1)} = .257$$

From Fig. B-1

$$\phi_2 = -.90 > -1.183$$

Step II

$$y_2 = .468$$

$$s/y_2 = \frac{.33}{.468} = .705, \ E/y_2 = \frac{.4962}{.468} = 1.06$$

$$F_2 = \sqrt{2(1.06-1)} = .346$$

From the Fig. B-1

$$\phi_2 = -1.25 < -1.183$$

Step III

$$y_2 = .472$$

$$s/y_2 = \frac{.33}{.472} = .70 \quad \frac{E}{y_2} = \frac{.4962}{.472} = 1.051$$

$$F_2 = \sqrt{2(1.051 - 1.0)} = .319$$

from Fig. B-1 $\phi_2 = -1.16 > -1.183$

Step IV

Assuming $y_2 = .47$

$$s/y_2 = \frac{.33}{.47} = .702, \quad E/y_2 = \frac{.4962}{.47} = 1.055$$

$$F_2 = \sqrt{2(E/y_2 - 1)} = \sqrt{2(1.055 - 1)} = .332$$

from Fig. B-1

$$\phi_2 = -1.20 < -1.183$$

By interpolation $y_2 = .4701$

$$Q_2 = B\, y_2 \sqrt{2g(E - y_2)}$$

$$= 2 \times .4701 \sqrt{2 \times 9.81\ (.4962 - .4701)}$$

$$= .635 \ m^3/sec$$

$$Q_s = Q_1 - Q_2 = .9 - .635 = .265$$

$Q_s = .265\ m^3/sec.$ Answer

COMPUTER SOLUTION (PART A)

GIVEN Y1, Q1, S, QS, B, G TO FIND L, Y2

```
C       N=NO. OF WEIRS
C       Y1=UPSTREAM DEPTH,Q1=UPSTREAM FLOW,S=HEIGHT OF WEIR CREST
C     L=LENGTH OF WEIR, B=WIDTH OF CHANNEL,G=ACCN DUE TO GRAVITY
        N=2
        AN=N
        REAL L
        PHI(F,ETA)=((2.+F**2.-3.*ETA)/(1.+.5*F**2.-ETA))*SQRT(.5*F**
       1(1.-ETA))-3.*ARSIN(SQRT(.5*F**2./(1.+.5*F**2.-ETA)))
        READ 10,Y1,Q1,S,QS,B,G
   10   FORMAT (6F7.2)
        F1=Q1/(Y1*B)/SQRT(G*Y1)
        V1=Q1/(Y1*B)
        E=Y1+V1**2./(2.*G)
        IF(F1-1.)20,20,30
   20   CM=.611*SQRT(1.-3.*F1**2./(F1**2 +2 ))
        GO TO 40
   30   CM=.36-.08*F1
   40   CONTINUE
        ETA1=S/Y1
        Q2=Q1-QS
        PHI1=PHI(F1,ETA1)
        A=Q2**2./(2.*G*B*B)
        Y2=Y1
        DO 80 J=1,25
        F2=SQRT (2.*(E/Y2-1.))
        ETA2=S/Y2
        YLAST=Y2
        FX=Y2**3.-E*Y2**2.+A
        FDX=3.*Y2**2.-2.*E*Y2
        Y2=Y2-FX/FDX
        IF(ABS(YLAST-Y2).LE..0001)GO TO 90
   80   CONTINUE
   90   CONTINUE
        PHI2=PHI(F2,ETA2)
        L=1.5*B*(PHI2-PHI1)/(CM*AN)
        PRINT 92, L,Y2
   92   FORMAT (//20X,*LENGTH OF THE SIDE-WEIR=*,F7.4,/20X,*DOWN
       1STREAM DEPTH=*,F7.4//////////////)
        STOP
        END
  SENTRY

   .44    .90    .33    0.15    2.00    9.81
```

EXECUTION

LENGTH OF THE SIDE-WEIR = 1.50 m
DOWNSTREAM DEPTH = 0.4627

COMPUTER SOLUTION (PART B)

GIVEN V1, Q1, S, L,B,G TO FIND QS,Y2

```
C       N=NO. OF WEIRS
C       Y1=UPSTREAM DEPTH,Q1=UPSTREAM FLOW, S=HEIGHT OF WEIR CREST
C     L=LENGTH OF WEIR,B=WIDTH OF CHANNEL,G=ACCN DUE TO GRAVITY
        N=2
        AN=N
        REAL L
        PHI(F,ETA)=((2.+F**2.-3.*ETA)/(1.+.5*F**2.-ETA))*SQRT(.5*F**2
       1(1.-ETA))-3.*ARSIN(SQRT(.5*F**2./(1.+.5*F**2.-ETA)))
        READ 10,Y1,Q1,S,L,B,G
   10   FORMAT(6F7.2)
        F1=01/(Y1*B)/SQRT(G*Y1)
        V1=Q1/(Y1*B)
        E=Y1+V1**2./(2.*G)
        IF(F1-1.)20,20,30
   20   CM=.611*SQRT(1.-3.+F1**2./(F1**2.+2.))
        GO TO 40
   30   CM=.36-.08*F1
   40   CONTINUE
        ETA1=S/Y1
        R1=L/B
        PHI1=PHI(F1,ETA1)
        PHI2=AN*R1*CM/1.5+PHI1
        Y2=Y1
        DO 80 J=1,25
        F2=SQRT(2.*(E/Y2-1.))
        ETA2=S/Y2
        YLAST=Y2
        FX=PHI(F2,FTA2)-PHI2
        FDX=(3.*Y2-2.*E)/(2.*SQRT((E-Y2)*(Y2-S)**3.))
        Y2=Y2-FX/FDX
        IF(ABS(YLAST-Y2).LE..0001)GO TO 90
   80   CONTINUE
   90   CONTINUE
        Q2=F2*B*SQRT(G)*Y2**1.5
        QS=Q1-Q2
        PRINT 92,QS,Y2
   92   FORMAT (//20X,*TOTAL FLOW OVER THE WEIR=*,F7.4,/20X,*DOWNSTREAM
       1DEPTH=*,F7.4//////////////)
        STOP
        END
 SENTRY
        .44    .90    .33    2.00    2.00    9.81
```

EXECUTION

TOTAL FLOW OVER THE WRIR = 0.2650
DOWNSTREAM DEPTH = 0.4701

APPENDIX C

HYDRAULIC JUMP ALONG THE SIDE-WEIR

APPENDIX C

HYDRAULIC JUMP ALONG THE SIDE-WEIR

C-1 Introduction:

In supercritical flow regime a hydraulic jump may fo: if the high velocity stream meets a subcritical stream of appropriate depth. It is possible for a hydraulic jump to ta place in the zone of a side-weir also.

Under such circumstances some length of the weir spills the flow which is supercritical in the channel and rest of the length passes out the subcritical flow after the jump. The discharge over the side-weir along its length is of highly varied nature, and does not render for easy solution.

An exploratory study of the hydraulic jump was carried out mainly on the weir of zero height and with only one experiment on the weir of finite height. In all these experiments, it was not possible to obtain hydraulic jumps with normal front. It was observed that the front of the jump was inclined at an angle θ' with the normal to the centre line of the channel. The value of θ' was observed to be approximately 45° for weirs of zero height and small Froude numbers. The value of θ' tended to decrease for high Froude numbers and finite height of weirs. No detailed study was done on the variation of θ'.

C-2 Analysis:

It was attempted to analyse the hydraulic jump occuring along the side-weir as an oblique jump. The value of θ' was assumed as 45°. The classical oblique jump equation (3) is;

$$\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{1+8F_1^{*2}}-1\right) \qquad \text{(C-1)}$$

where

$$F_1^{*} = F_1 \cos \theta'.$$

For the experimental data y_2/y_1 has been plotted against F_1^{*} in fig. C-1. Also plotted is equation (C-1). It is seen that for low Froude numbers the experimental data follow closely the plot of equation (C-1). However, for high Froude numbers the points diverge up, leading to the conclusion that θ' decreases with increase in Froude number. The value of θ' is also decrease for finite height of weirs. It is felt that the variation of θ' should be expressible as

$$\theta' = fn'(F_1, s/y_2) \qquad \text{(C-2)}$$

To find the exact form of equation (C-2) a detailed study is needed.

It was concluded that the hydraulic jump occuring along the side-weir cannot have a normal front. It could be analysed as an oblique jump (Eq. C-1). However, a detailed study is needed to express inclination θ' of the front in terms of the flow parameters, i.e. F_1 and s/y_2.

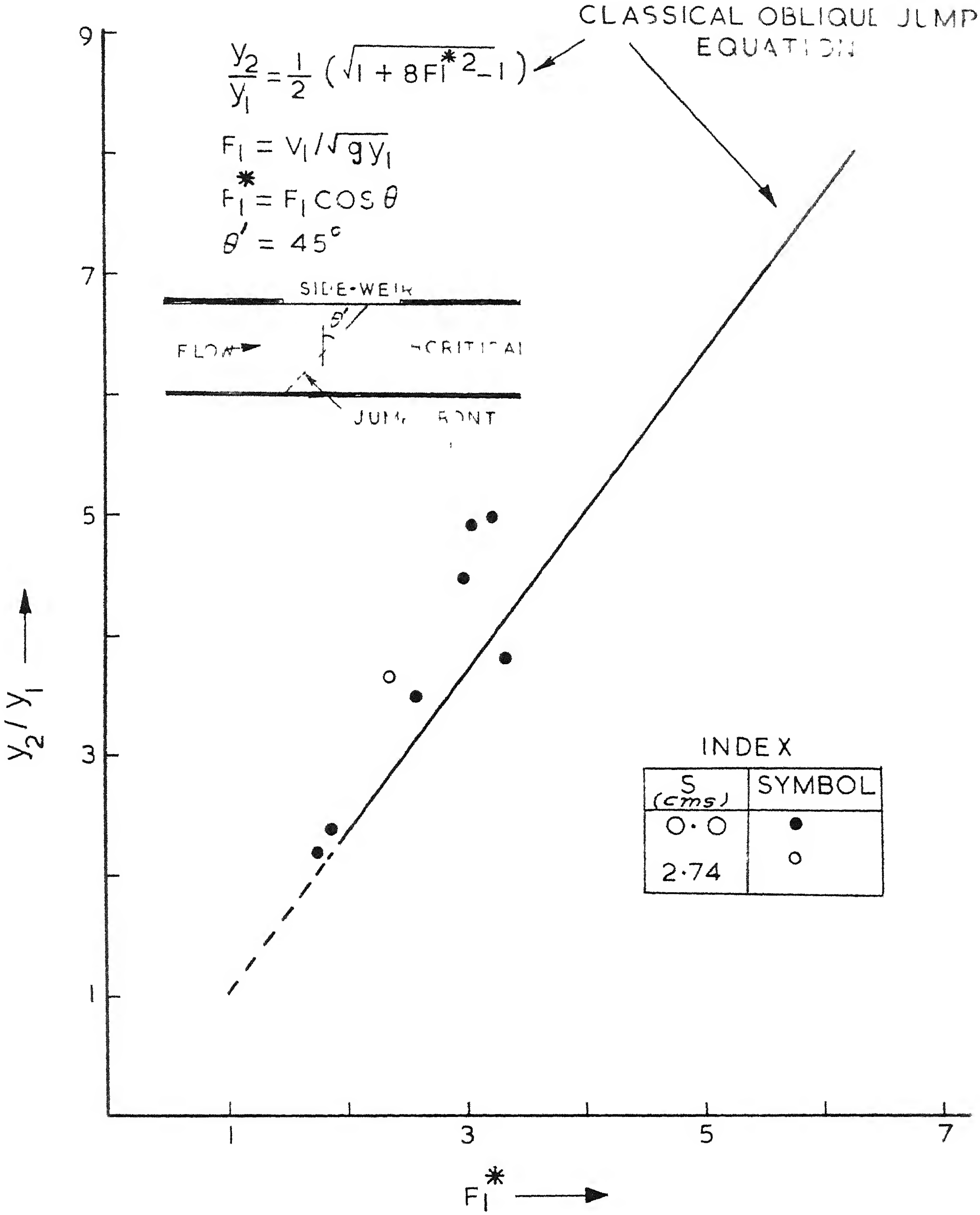

FIG. C-1 HYDRAULIC JUMP ALONG A SIDE-WEIR

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Hydraulic behavior
of side-weirs.